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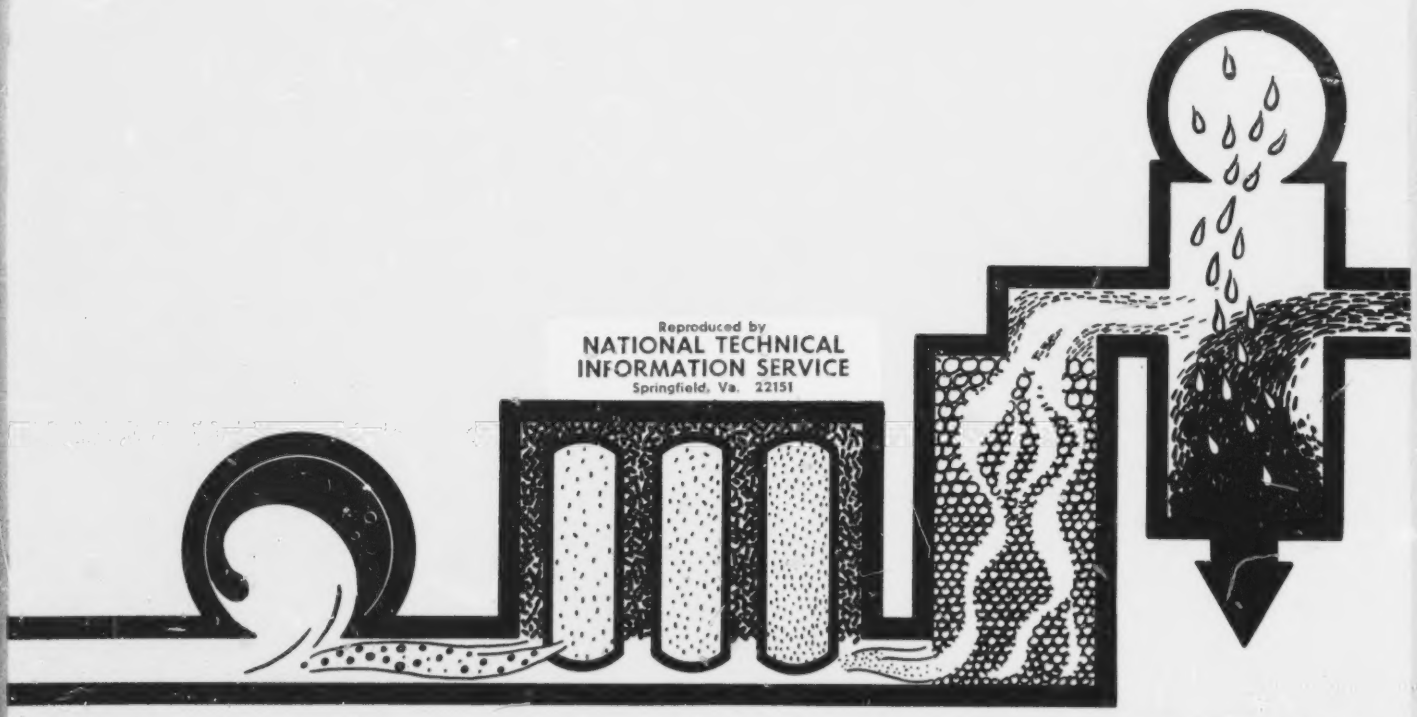
A Mathematical Model of a Final Clarifier



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16. Abstract: Final clarifiers at three treatment plants were subjected to experimental testing to provide a set of data for the formulation and testing of a mathematic model for a final clarifier. Multiple regression analysis techniques were employed to develop equations for estimating the return sludge and effluent suspended solids concentrations. Those equations developed were: (1) maximum return sludge concentration (mg/l) = $10^6/540 \times A^{4.397} \times B^{0.213}$, where A = fraction of volatile suspended solids and B = BOD loading (lbs BOD/day/lb MLVSS); and (2) effluent suspended solids (mg/l) = 38[illegible] x (overflow rate)$^{0.12}$ x $B^{0.27}$/MLSS$^{0.35}$ x (Detention time)$^{1.03}$. Results of this investigation demonstrated that neither return sludge concentration nor effluent suspended solids concentrations could be estimated with any reasonable accuracy from those parameters normally available to the design engineer. Future investigations must include the development of some method for estimating the solids settling rate based on solids characteristics and activated sludge operational parameters before any further progress in mathematical modeling can be made. (Lowry-Texas)

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Abstractor: Mark V. Lowry | Institution: Texas Univ, Austin

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A MATHEMATICAL MODEL OF A FINAL CLARIFIER

by

Rex Chainbelt, Inc.
Milwaukee, Wisconsin 53201

for

Office of Research and Monitoring

ENVIRONMENTAL PROTECTION AGENCY

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ABSTRACT

The final clarifier in the activated sludge process performs a vital role in secondary waste treatment systems. The final clarifier must perform the dual function of providing an effluent low in suspended solids and must be capable of providing an underflow of sufficient concentration to permit the maintenance of a suitable population of active microbial mass in the aeration tank. The purpose of this project was to develop a mathematical model to predict the solids concentration of both the underflow and overflow of a final clarifier as a function of the mixed liquor characteristics. This model was to utilize, as much as possible, those parameters which are normally available to engineers involved in the design of final clarifiers.

An experimental testing program was carried out on final clarifiers at three treatment plants in order to provide a set of data for the formulation and testing of the model. The techniques of multiple regression analysis were used to develop the following equations for estimating the return sludge and effluent suspended solids concentrations.

$$\text{Maximum Return Sludge Concentration (mg/l)} = \frac{10^6}{540 \times A^{4.397} \times B^{0.213}}$$

where: A = fraction of volatile suspended solids
B = BOD loading lbs BOD/day/#MLVSS

$$\text{Effluent Suspended Solids (mg/l)} = \frac{382 \times \text{Overflow Rate}^{0.12} \times \frac{(\#\text{BOD/day})^{0.27}}{\#\text{MLSS}}}{\text{MLSS}^{.35} \times \text{Detention Time}^{1.03}}$$

The effluent solids equation had a multiple correlation coefficient of 0.63. The magnitude of the correlation coefficient for the effluent solids equation was lower than anticipated because of changes in sludge quality, both interplant and intraplant which were not accounted for by the parameters considered in this study.

Based on the results of this work it can be concluded that neither the effluent suspended solids nor the return sludge concentration can be estimated with good accuracy from those parameters which are normally available to the design engineer.

Recommendations for future work which would enable a more realistic approach to the preliminary design of final clarifiers are made. For example, techniques for estimating the sludge subsidence characteristics in terms of the operational parameters of the activated sludge system need to be developed because generally only the operational parameters are available for preliminary design and simulation studies.

This report was submitted in fulfillment of Project Number 17090FJW, Contract 14-12-194, under the sponsorship of the Environmental Protection Agency.

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CONCLUSIONS

1. The maximum return sludge concentration from a final clarifier can be estimated from the relationship:

$$TSS_6 \text{ (mg/l)} = \frac{10^6}{540 \times A^{4.397} \times B^{0.213}}$$

where: A = fraction of volatile suspended solids
B = BOD loading - lbs BOD/day/lb MLVSS

2. The effluent suspended solids from a final clarifier can be estimated from the relationship:

$$C_E = \frac{382 \times \text{Overflow Rate}^{0.12} \times \text{BOD Loading}^{0.27}}{\text{MLSS}^{.35} \times \text{Detention Time}^{1.03}}$$

3. The relationship for effluent suspended solids needs to be improved. Based on information collected in this study a new parameter, sludge quality, is proposed which would account for the various classifications of solids comprising activated sludge (active cells, inert cells, digested fines, etc.) and the ability of these solids to flocculate.

4. Future studies of activated sludge systems should include an evaluation of the sludge subsidence characteristics and sludge quality as a function of the operational parameters of the activated sludge system.

5. The most pressing need in the development of sedimentation models is a formula relating the settling rate of activated sludges to the operational characteristics of the aeration tank.

6. Commonly accepted parameters of settling rate and overflow rate are gross measurements which are not sensitive enough to adequately predict final clarifier performance. They are, however, useful as design and operational parameters.

RECOMMENDATIONS FOR FUTURE RESEARCH

There is a great necessity for a mathematical model to predict the performance of final clarifiers. Reliable estimates of solids output from final clarifiers are needed to design tertiary treatment systems, as input to river basin treatment optimation studies and for in-plant optimization studies. In order to improve the reliability of the equations presented in this report and to permit preliminary design, the following items need further investigation.

1. A more reliable laboratory procedure that will closely approximate the settling rate in full-sized clarifiers needs to be developed. Particular emphasis should be given to cylinder diameter, cylinder depth and mixing.

2. A method to permit the prediction of the settling rate of the activated sludge as a function of the solids characteristics and the operational parameters of the activated sludge system needs to be developed. This procedure would then permit the preliminary design of final clarifiers based on a knowledge of the sludge to be separated.

3. The sludge quality parameter as defined in this report needs to be evaluated to determine the effect of the various classifications of solids comprising the sludge on the effluent quality from the clarifier.

4. Side by side tests of various types of final settlers need to be made to evaluate the effect of geometric configuration on final effluent quality. It is only when two clarifiers are evaluated while handling the same sludge that meaningful conclusions can be drawn concerning the effect of tank geometry and inlet and outlet configurations.

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INTRODUCTION

The increased demands for the abatement of water pollution in the United States has resulted in an increased interest in mathematical modeling of the unit processes in the waste treatment field. Mathematical models are useful both as an aid in the understanding of process behavior and in optimizing a system composed of a number of unit processes.

The purpose of this project has been to develop a mathematical model to predict the performance and design requirements of a final clarifier in the activated sludge process. An attempt has been made to use parameters in the model which are routinely available to consulting engineers. It is felt that this approach is necessary in order to make the model of practical use and not just a theoretical study.

Since the effluent of the final clarifier represents the quality of treatment achieved at most treatment plants prior to its discharge into the receiving waters, the final clarifier might be considered the most important unit process in a secondary waste treatment plant.

The final clarifier in the activated sludge process has two major functions. It must discharge an effluent (overflow) which is low in suspended solids. At the same time, it must be capable of removing settled activated sludge (underflow) at a sufficiently high concentration to maintain a satisfactory inventory of viable solids in the aeration tanks. In addition to the very important considerations of the activated sludge settling rate, the general hydraulic parameters, and the clarifier design, the sludge retention time and distribution within the clarifier are of utmost importance.

It must be realized that an activated sludge system is an integral treatment process consisting of both an aeration and a sedimentation step. This interrelation must always be considered when either step is under study. For example, under certain conditions of deficient aeration, a "bulking" activated sludge may be produced. This sludge, which exhibits very poor settling characteristics, may prevent the final clarifier from producing an effluent which is low in suspended solids. Under normal operation, with sufficient aeration, the sludge produced in the aeration tanks may readily be settled out in the associated final clarifiers.

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LITERATURE SEARCH

The sedimentation process has been the subject of much research and design effort. A literature search has been made in an attempt to assemble the currently accepted theories of the final clarification of activated sludge in order to elucidate the methods and procedures to be followed in fabricating a mathematical model to predict the performance of a final clarifier. The literature search has been narrowed to include the topic of sedimentation only as it applies to activated sludge.

The first section of this review deals with the sludge solids and their relationship to the overall process. The second section discusses the design requirements and the operational characteristics of final clarifiers.

Characteristics of the Solids

Katz et al(1) have divided suspensions into three general classifications:

Class I - Discrete particles, which will not readily flocculate and which predominate in relatively low concentrations. An example of this type of suspension is encountered in grit chamber design and in clarification of certain industrial wastes such as sand and gravel washings, etc.

Class II - Relatively low solids concentrations of flocculent material. An example of this type of material is found in primary settling tank influents, water which has been subjected to flocculation and numerous industrial wastes.

Class III - Encompasses materials of relatively high concentrations. The material may be flocculent, but not necessarily so. Hindered settling is the term generally used to describe separation of this type of solids. Examples of this type of separation are found in activated sludge settling and industrial wastes, such as paper and pulp.

Class I and II suspensions are not normally encountered in final clarifiers and hence will not be further considered here.

The settling process of Class III suspensions has been described by Eckenfelder and O'Connor(2) and is shown in Figure 1. During the initial settling period (A) the sludge floc settles at a uniform velocity under conditions of zone settling. The magnitude of this velocity is dependent on the initial solids concentration. The concentration of solids during this period remains constant until the settling interface approaches an interface of critical concentration. With an increase in depth of the settled sludge, the floc begins to press on the layers below and the

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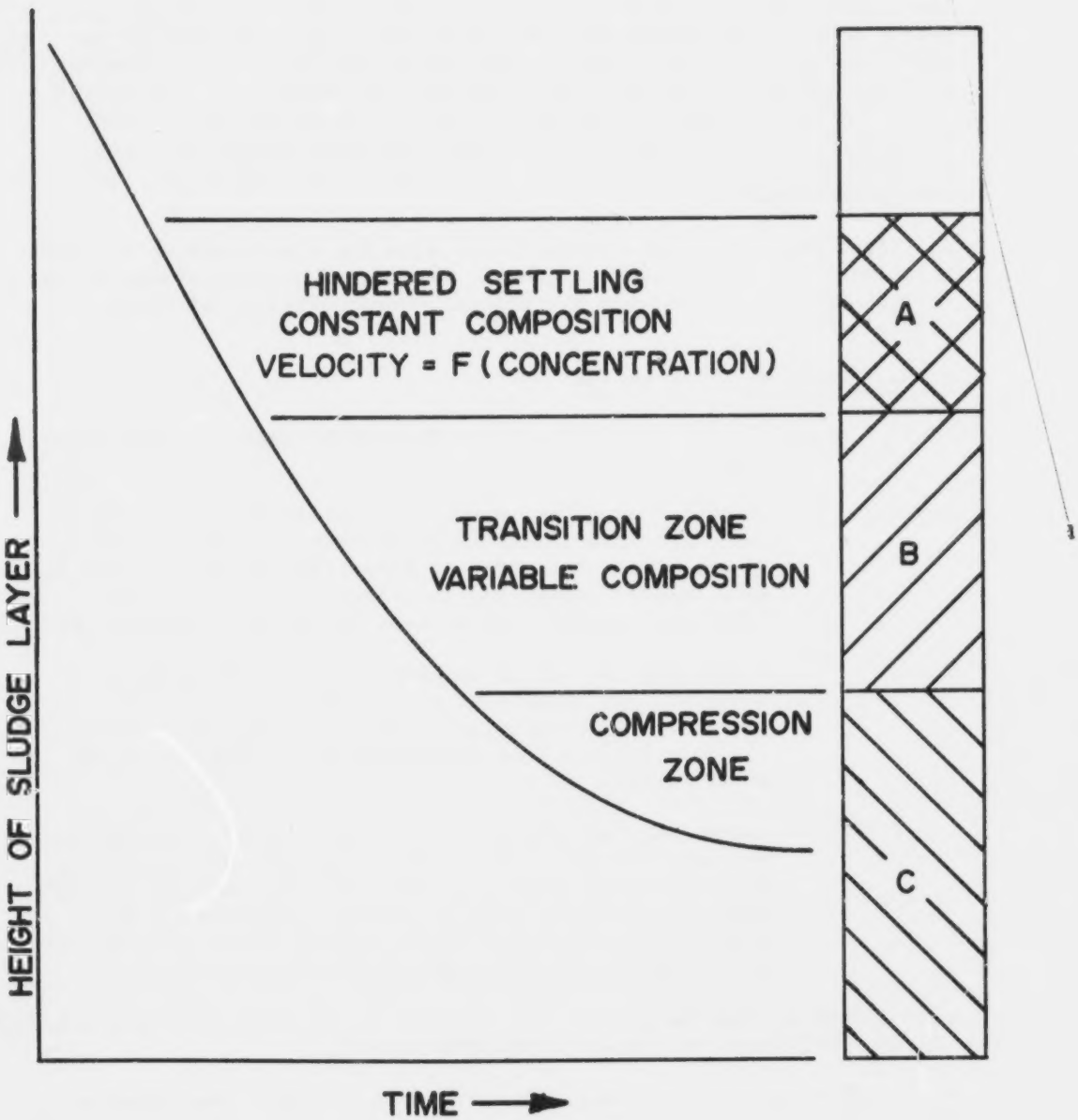


FIGURE 1

SETTLING ZONES FOR CLASS III SUSPENSIONS

transition zone occurs. The settling velocity decreases in the transition zone due to the increasing density and viscosity of the suspension surrounding the particles. A compression zone (C) occurs when the floc concentration becomes so great as to be mechanically supported by the layers of the floc below. The solids concentration in the compression zone is related to the depth of the sludge and the detention time of the solids in this zone.

Activated Sludge Modifications

The activated sludge process can be and is operated over a broad spectrum of growth phases ranging from high rate - dispersed growth to extended aeration systems. The growth phase or physiological state of the microorganisms has been implicated as an important factor in the separation of the activated sludge solids from water (3)(4). An idealized growth curve for the various activated sludge modifications has been presented by Lesperance(5).

High rate activated sludge systems operate in the log growth-declining growth phases and the solids tend to be difficult to separate in gravity separation systems. Their primary use is found in areas where a high quality effluent is not required for discharge to the receiving waters.

Conventional activated sludge is probably the most commonly used method of treatment. The systems are operated in the declining and endogenous growth phases with BOD loadings up to about 0.5 lb.BOD/day/lb.MLSS. Most of the difficulties reported in the literature concerning activated sludge treatment have occurred in the conventional systems.

The step aeration process was developed in an effort to overcome some of the problems associated with the conventional activated sludge process. In this method, the raw waste is introduced at a number of points along the length of the aeration tank. The process tends to stabilize the growth phase within a narrow range as compared to wide fluctuations in the conventional process. In addition, it allows a savings in aeration tank volumes(6). The gross BOD loading would be of the same order of magnitude as the conventional system.

A more recent development, complete-mixing activated sludge, has been advocated by a number of people(7)(8). In this modification, the raw waste is intimately mixed with the activated sludge solids to maintain a uniform BOD and mixed liquor solids loading in the entire tank. Proponents of the process feel that steady state organic loading and biological growth characteristics can best be maintained in this type of system.

Numerous other activated sludge modifications exist, each having merit under certain conditions. These modifications have arisen primarily as a result of plant operators' efforts to solve a particular problem. For example, the Kraus modification was developed in an attempt to overcome a bulking sludge which was very difficult to separate.

Extended aeration is the extreme process modification on the low organic loading scale. Aeration detention times of 24 hours are normally maintained at BOD loadings of 0.1 to 0.2 lb.BOD/day/lb.MLSS. Final clarification of the mixed liquor from these systems often results in poor quality effluent resulting from denitrification or pinpoint floc resulting from overaeration.

A summary of the design and operational parameters for the various activated sludge modifications is shown in Table 1. Although there is little or no data in the literature relating the solids subsidence characteristics of activated sludges to the various process modifications there can be little doubt that some relationship does exist.

Ford and Eckenfelder(4) reported on the results of literature studies in which three industrial wastes were studied over a range of organic loadings. These results are shown in Figures 2, 3, and 4. It can be seen that increases in organic loadings tend to decrease the subsidence and compaction characteristics of the sludges. For these wastes, the optimum loading from a solids separation standpoint was about 0.3 to 0.4 lb.BOD/day/lb.MLSS.

Based on the data available in the literature, it would appear that additional investigation of the effect of organic loadings on the subsidence characteristics of sludges would be fruitful.

Biological Factors

The development of an activated sludge depends on a number of parameters including the waste characteristics, growth rate of the micro-organisms and the availability of the essential nutrients. Depending on these variables, the sludge may range from predominantly bacteria to filamentous bacteria to fungi. Since the overall performance of secondary waste treatment is primarily a function of the solids separation which occurs in the final settler, the predomination of various types of micro-organisms becomes an important consideration.

An activated sludge developed from a nutritionally balanced waste is generally predominantly bacterial in composition with some protozoa and higher forms of life. Bacterial sludges generally have good subsidence characteristics.

Wastes high in carbohydrates, or low in nitrogen have been shown to produce filamentous type sludges. Data shown in Figures 2, 3, and 4 indicate that organic loadings in excess of 0.5 lb.BOD/day/lb.MLSS also tend to promote filamentous type sludges. Filamentous sludges tend to be difficult to settle because of their large surface area to volume ratio and their low density.

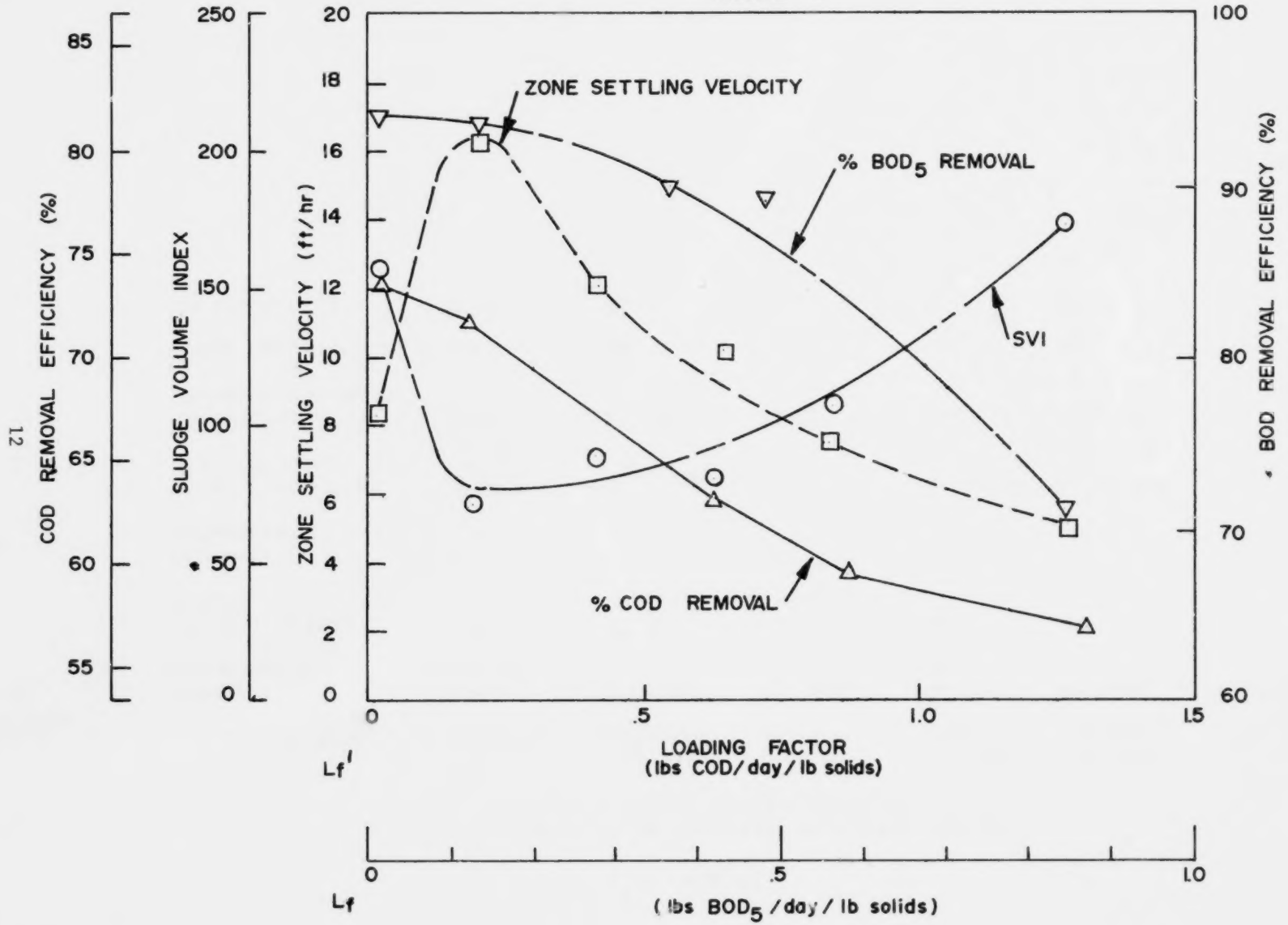
TABLE 1

A Comparison of the Biological Characteristics of Various Activated Sludge Processes(61)

Characteristics	Conventional	High Rate or Modified	Contact Stabilization	Extended Aeration	Complete Mixing
Primary Sedimentation	Usually Provided	Optional	Optional	Generally None	Optional
Aeration Period (hours)	5-10	2-3.5	0.33-0.67 (contact)	24	2
Secondary Sedimentation	Yes	Yes	Yes	Yes	Yes
Return Sludge Flow (% of Raw Flow)	25-50	10	30-50	up to 100	up to 100
BOD Loading (lb/day/100 lb MLSS)	25-50	200-400	15-35	about 15	about 60
Sludge Age (days)	3-6	1/4-1/2	3-7	>10	1-2
BOD Removal (%)	85-90	60-75	90	98	85-90

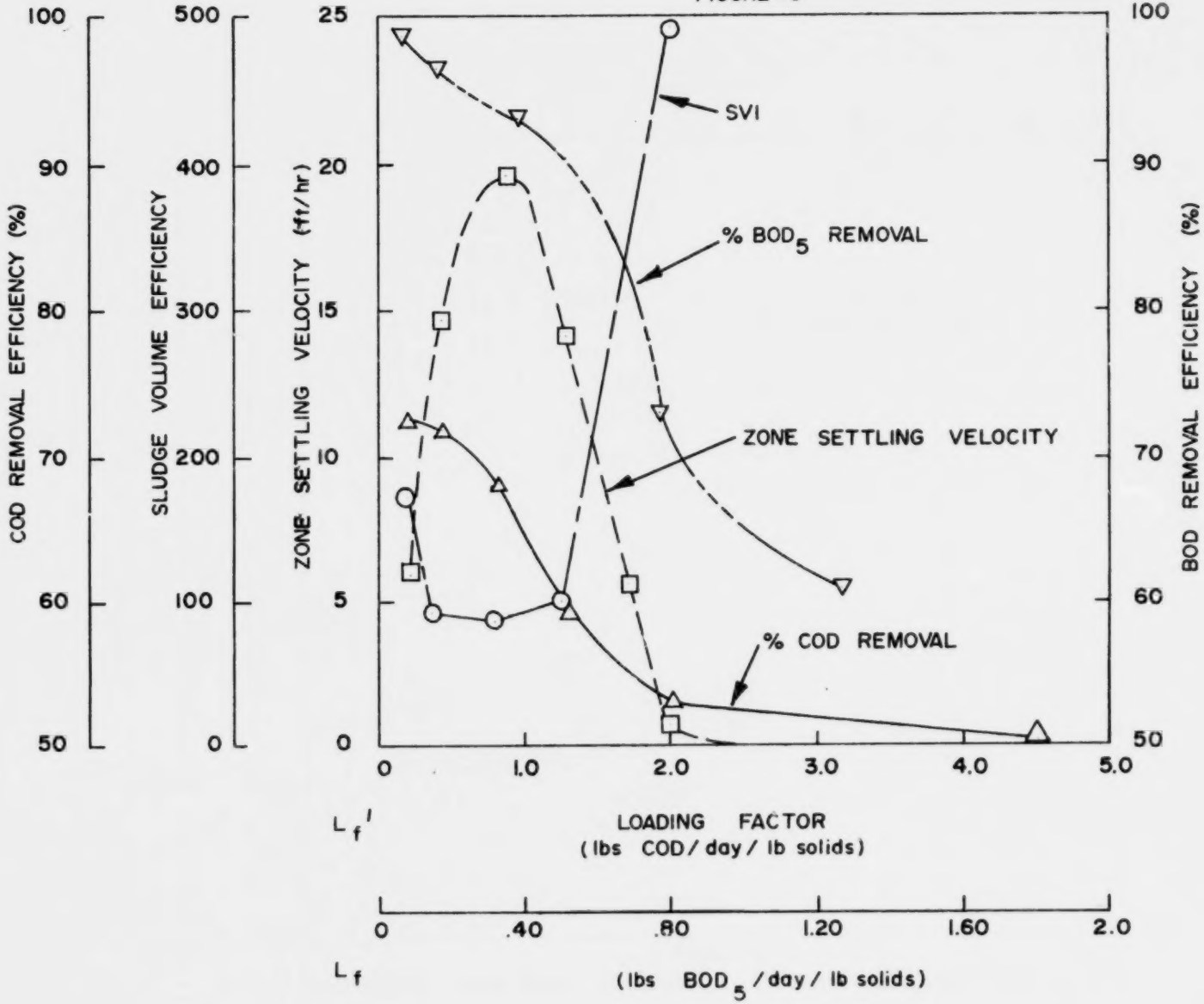
PARAMETER RESPONSE TO ORGANIC LOADING, DOMESTIC WASTE

FIGURE 2



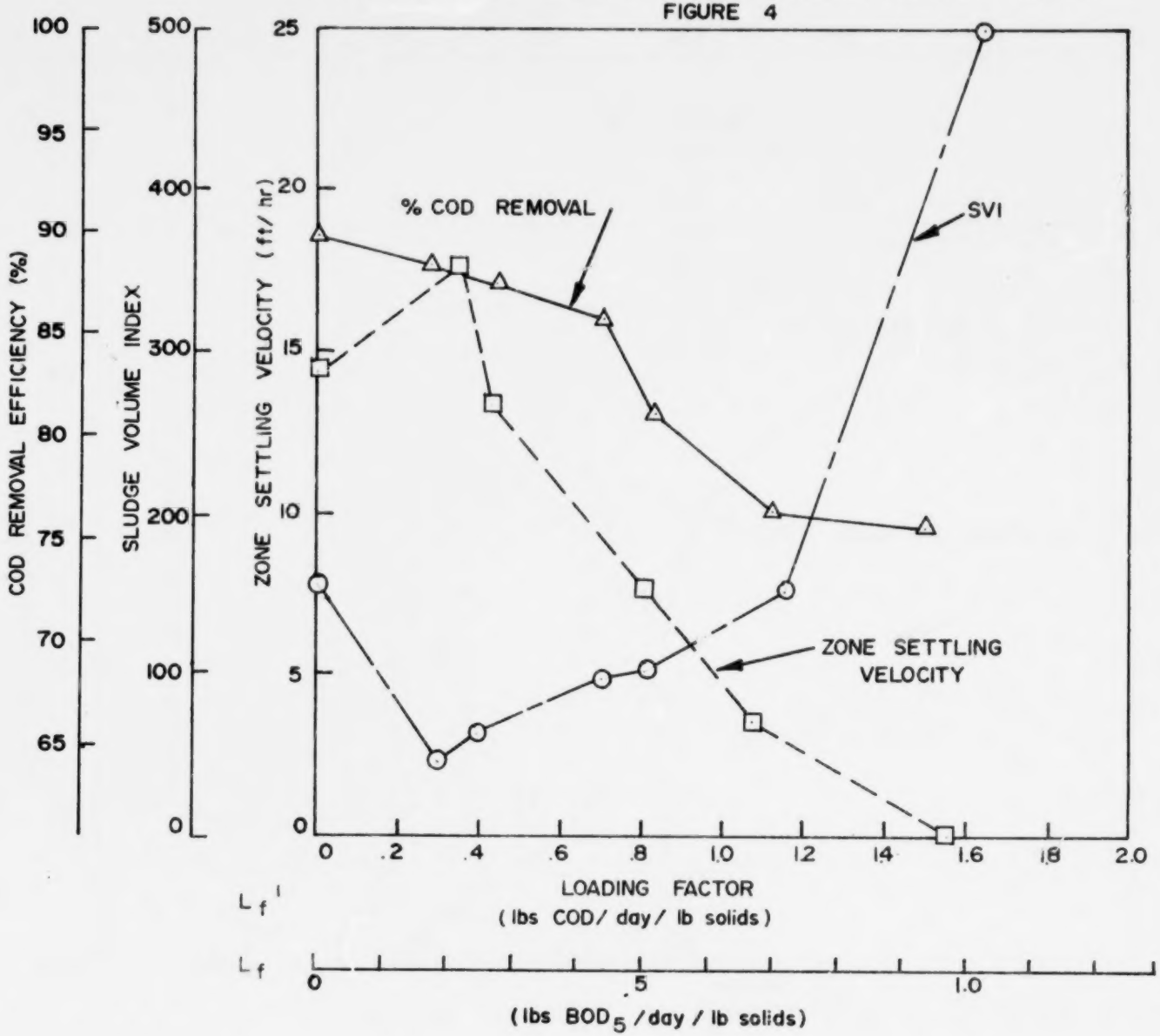
PARAMETER RESPONSE TO ORGANIC LOADING, PETROCHEMICAL WASTE

FIGURE 3



PARAMETER RESPONSE TO ORGANIC LOADING, BREWERY WASTE

FIGURE 4



The environmental conditions in the aeration tank are important to the development of a sludge with good subsidence characteristics. The environmental conditions affecting sludge characteristics include dissolved oxygen concentration, pH, sludge age, and intensity of aeration. Pipes(9) has discussed various types of activated sludges and attempted to classify sludges according to whether they bulked or not. The basic classifications presented by Pipes together with their apparent causes are shown in Table 2. This classification clearly indicates the intimate relationship between the environmental conditions in the aeration tank and the design and performance of the final clarifier.

The various types of sludges and their related causes will not be discussed in detail because the majority of the causes have not been proven. As Pipes pointed out in his discussion, much work needs to be done to improve the understanding of the causes of solids separation problems in activated sludges.

Settling Characteristics of Solids

The settling rate of mixed liquor solids is normally obtained by observing the position of the water-solids interface as the solids settle in a one liter graduate. The settling rate is then determined as the slope of the line in the free settling zone expressed in units of feet per minute or feet per hour.

When sludge samples are available prior to the design of a sedimentation tank, this settling rate is used in the design procedure. The settling rate can be converted to an upflow velocity expressed in gallons per day per square foot. This upflow velocity must always be greater than the design overflow rate.

The concentration of mixed liquor solids is known to affect the settling rate of the solids (1)(10). This relationship is shown in Figures 5 and 6.

In Figure 5 the initial settling velocity is plotted versus initial depth for various MLSS concentrations. The settling velocity decreases linearly with increases in MLSS. Figure 6 is a plot of settling rate versus initial solids concentration. The decrease in settling rate with increased MLSS is similar to that shown in Figure 5.

The effect of gentle mixing on the settling rate can also be seen. The effect of mixing becomes more beneficial at high MLSS concentrations. Dick and Ewing(10) also observed the benefits derived from gentle mixing which can influence the zone settling velocities and increase the solids transmitting capacity of activated sludge.

A number of mathematical expressions have been presented in the literature which relate mixed liquor suspended solids concentrations to the settling velocity of activated sludges. These equations are in most cases entirely

TABLE 2

Classification of Various Difficult-to-Separate Activated Sludges

Classification	Probable Cause
I. Bulking Sludge	
a) Non-Filamentous Bulking	Presence of large quantities of extracellular materials with a high degree of hydration producing a sludge with excessive amounts of bound water.
b) Filamentous Bulking	The predomination of fungi; as a result of certain environmental factors, i.e., low pH, low dissolved oxygen.
II. Rising Sludge	Denitrification in the sludge blanket.
III. Septic Sludge	Excessive sludge detention times in the final clarifier resulting from poor clarifier design.
IV. Overaerated Sludge	Excessive aeration causes bubbles to be carried into the final clarifier and causes the sludge to be buoyed to the surface by the rising bubbles.
V. Floating Sludge	Presence of sludge particles whose density is less than water.
VI. Pinpoint Floc	Excessive turbulence in the aeration tank.
VII. Billowing Sludge	Hydraulic surges, density currents, stirring by sludge scrapers.

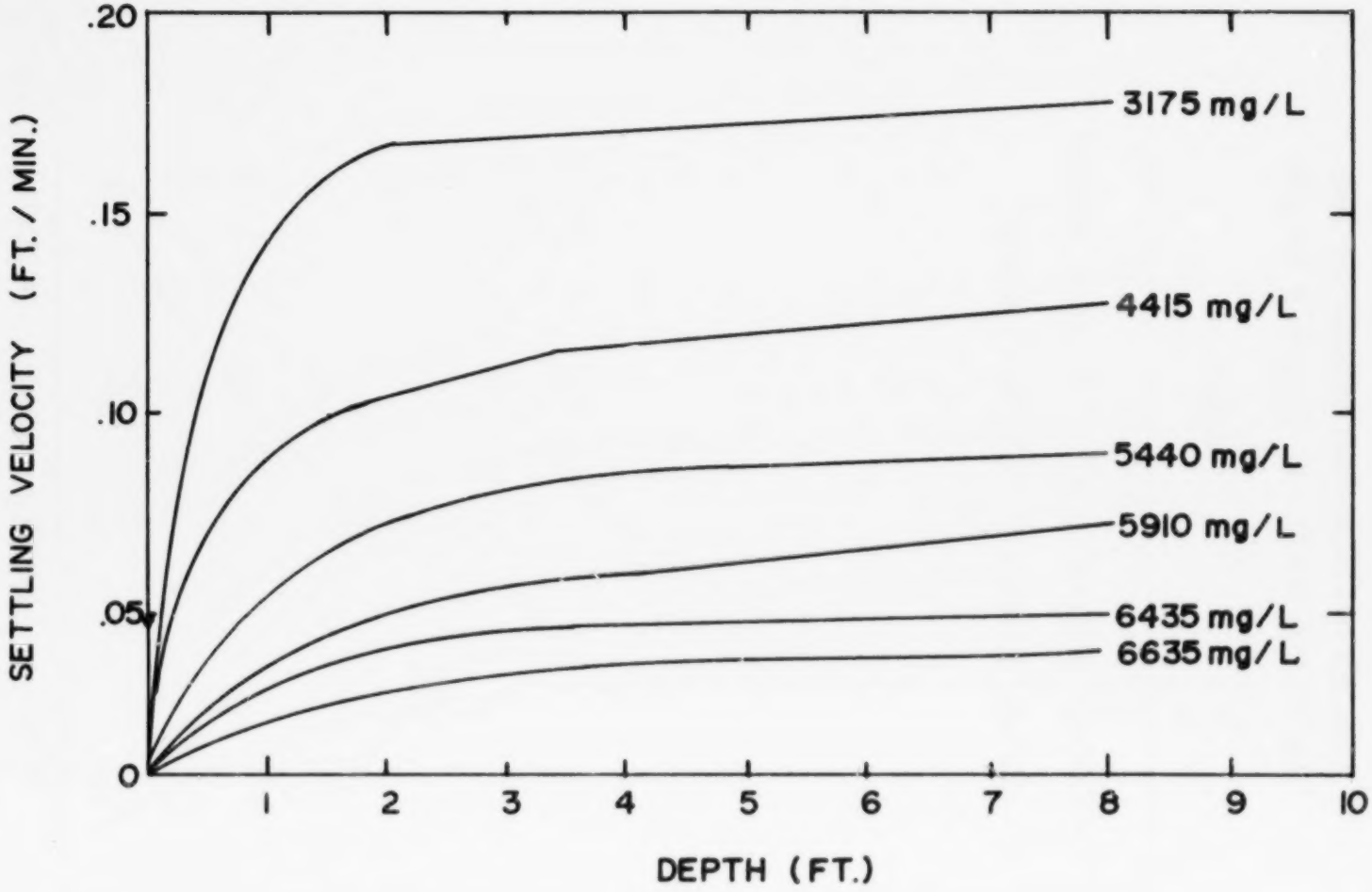
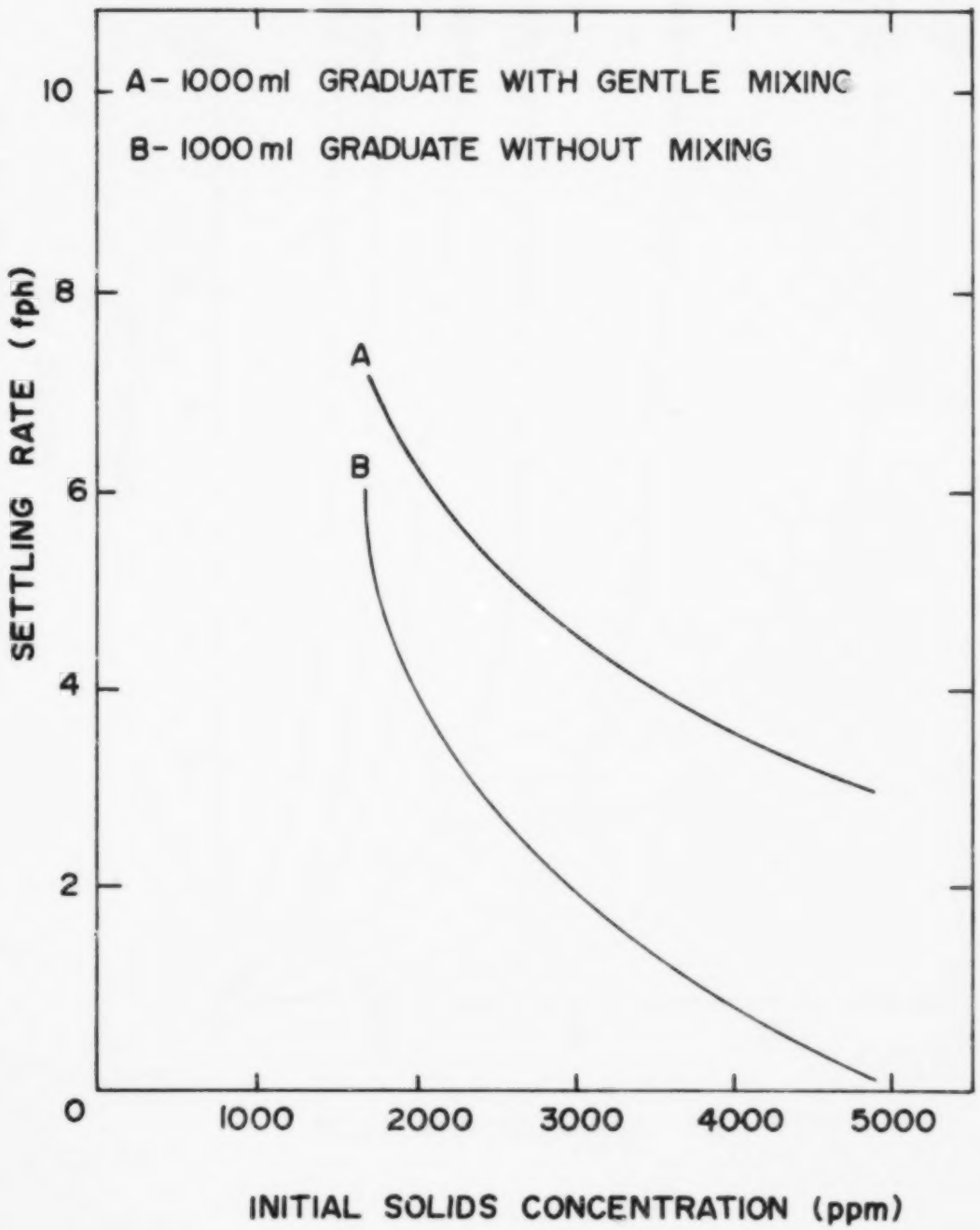


FIGURE 5

SETTLING CHARACTERISTICS OF AN ACTIVATED SLUDGE

THE INFLUENCE OF INITIAL SOLIDS CONCENTRATION AND MIXING ON THE SETTLING RATE OF CLASS 3 SOLIDS

FIGURE 6



empirical and may result in serious error when used for sludges other than those for which they were developed. These equations are presented in Table 3 to give an estimate of the general form of the equations.

In addition to concentration, settling column diameter and initial sludge depth have been shown to affect the subsidence rate of activated sludges (10)(11). Vesilind(11) presented a particularly interesting curve (Figure 7) which relates the diameter of the test cylinder to the relative settling velocity for various mixed liquor suspended solids concentrations. These curves, although specific for the sludges tested, clearly indicate the effect of column diameter on batch settling rates . For suspended solids concentrations less than about 5000 mg/l, small diameter settling columns yield settling rates greater than might be expected in a final clarifier. Vesilind attributed this increased velocity to wall effects. For suspended solids concentrations greater than about 5000 mg/l, settling rates less than what might be expected in full scale units were found to occur. This effect is thought to be caused by bridging of the sludge in the small diameter cylinders. Dick and Ewing(12) confirmed this observation and suggested that settling properties of activated sludges be investigated in columns as large as practically possible. Mancini(13) reported that cylinder diameters up to 12 inches had no effect on the initial sludge settling rate for the sludges he tested.

Based on the curves presented by Dick and Ewing(10), it would appear that the variation in sludge settling velocity with depth is important at small initial depths such as occur in laboratory tests (Figure 5). For initial sludge depths greater than about six feet, very little change in settling rate was observed with increased depths.

The effect of temperature on the settling rate of activated sludge and sewage has been discussed by Rudolfs and Lacey(14) and Ridenour(15). The rate of settling of sewage solids has been shown to increase with an increase in temperature up to about 30°C(15). For settling periods longer than 30 minutes, this difference in settling rate had no significant effect on the settling efficiency. The settling rate of activated sludge was found to decrease for decreases in temperature(14). The difference in settling rates at low temperatures may be partially explained by the slower rate of sludge oxidation and flocculation which may occur at low temperatures. The difference might also be accounted for by an increase in the density of the liquid medium at lower temperature, thus lowering the driving force for sedimentation. Pflanz(16) presented data which indicated that the effluent suspended solids of a final clarifier increased from 1.5 to 2 times at similar surface loading rates (Kg/m^2hr) as the temperature decreased from approximately 14°C to 20°C.

Sedimentation tanks may also be affected by thermal gradients resulting from differences in temperature between the sedimentation tank contents and the influent mixed liquor flow. Hall(17) has attributed short-circuiting in sedimentation tanks to temperature gradients.

TABLE 3

Mathematical Expressions for Settling Rate as a Function of Solids Concentration

Equation Presented By	Equation	Definitions of Variables
Krone(62)	$V = V_o(1 - KC)^{4.65}$	V = Group settling velocity V_o = Settling velocity of individual aggregates C = Initial concentration of suspended solids K = Volume of aggregate/gram of solids
Duncan and Kawata(63)	$V = ac^b$	V = Initial settling rate c = Initial solids content b = Empirical constant a = Sludge constant
Vesilind(11)	$V = V_o e^{-kn}$	V = Initial settling rate V_o = Experimentally determined settling rate at cencentration c c = Sludge concentration k = Sludge constant

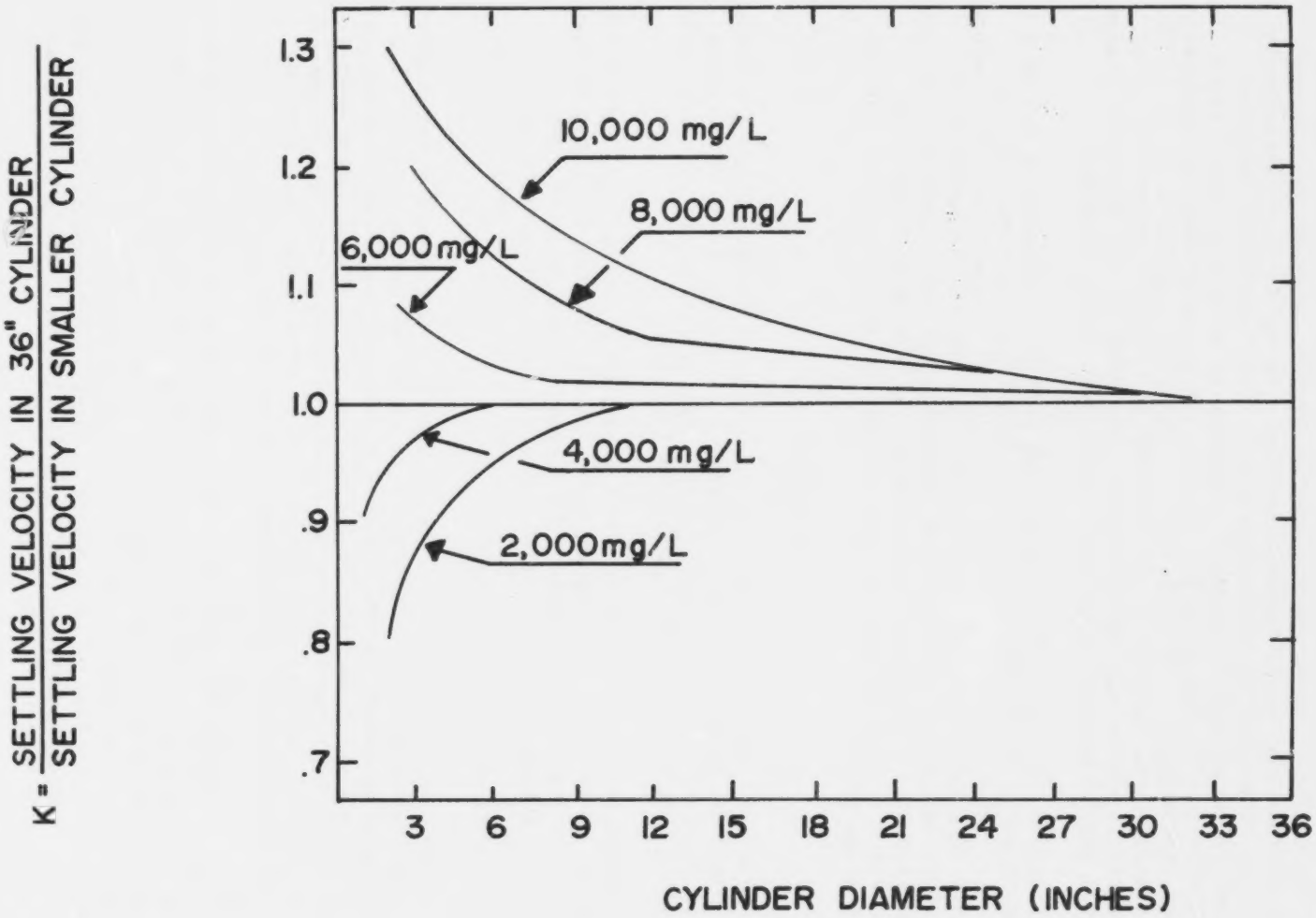


FIGURE 7 EFFECT OF SETTLING COLUMN DIAMETER ON BATCH SETTLING RATE

Flocculation of bacteria is essential to the operation of the activated sludge process. Without flocculation, the bacteria remain dispersed and are difficult to separate from the liquid. Camp(18) has proposed that flocculation in a sedimentation basin is due to 1) differences in settling velocities of particles wherein "fast" particles overtake "slow" particles and 2) velocity gradients in the liquid which cause particles in a region of higher velocity to overtake those in adjacent lower stream paths. Although they are not often discussed in the literature, such factors as the physical and chemical properties of the bacterial surfaces and the biological properties of the activated sludge have a significant effect on the flocculation of the bacteria.

McKinney(3) has stated that the average size of a bacteria is 0.5 to 3.0 microns which is slightly larger than colloidal particles (.001 to 1 micron). Riddick(19) states that particles of a size from 1 to 10 microns behave in a manner similar to colloids. Because of the presence of electric charges on their surface, colloidal particles possess a certain stability or resistance to flocculation. This stability has been attributed to the magnitude of the zeta potential (ζ) which is defined by the equation:

$$\zeta = 4\pi\delta q/D$$

In which q is the charge on the particle (or the charge difference between the particle and that of the solution), δ is the thickness of the layer round the particle through which the charge difference is effective and D is the dielectric constant of the medium (20).

Zeta potential is used in some water treatment plants(19) to control the chemical dosages for the coagulation of water. Schroepfer(21) felt that under normal conditions, electrical charges on particles of suspended matter in sewage do not influence their rate of sedimentation to any great extent. No mention is found in the literature of zeta potential measurements on activated sludge.

The close relationship between the operation of the aeration tanks and the performance of the final clarifiers cannot be overemphasized. The operational parameters of BOD loading, sludge age, and dissolved oxygen concentration all contribute to the quality of the mixed liquor solids. These parameters together with the raw waste characteristics determine the biological predomination and hence provide a significant contribution to the subsidence characteristic of the sludge.

Garrison and Nagel(22) have shown that low sludge volume indexes result in powdery, pinpoint floc which is carried out in the effluent. High SVI's have poor settling characteristics and require the handling of large quantities of return sludge. They reported that the SVI could be controlled by the quantity of air supplied, reaeration of the return sludge and by control of the organic loading. High organic loadings are generally associated with high SVI's. Similar conclusions can be drawn from the work reported by Dye(23).

Keefer(24) reported an improvement in the quality of the effluent from final clarifiers with the presence of a bulking sludge. Superimposed on this phenomenon is the requirement of adequate capacity in the final sedimentation tanks and return sludge pumps. Figure 8 shows this relationship for various temperatures and overflow rates. Increases in SVI above that shown in Figure 8 tended to increase the effluent suspended solids. It would appear thus that there exists an optimum overflow rate and SVI level for maximum suspended solids removal.

Factors Affecting Sludge Thickening

The design of final clarifiers must also consider the concentration of suspended solids in the underflow. Economic waste treatment design would dictate that the underflow solids be concentrated as much as possible, consistent with good clarification and economic tank design.

Most of the modern thickening procedures are based on the work of Coe and Clevenger(25). They proposed that each concentration of a suspension has a certain capacity to discharge its solids. This capacity is given by:

$$C = V[(1/C_i) - (1/C_u)]$$

In which C - capacity of the suspension at concentration C_i, to transmit solids if the suspension is being thickened to a concentration C_u. The settling velocity is represented by V.

The authors pointed out that if a layer in a suspension has a lower solids-handling capacity than an overlying layer, it will not be able to discharge solids as fast as they are being received and the solids layer will build up. Similarly, if a layer is able to transmit solids at a faster rate than they are received from the overlying area, its thickness will remain infinitesimal. Design should then be based on an area sufficiently large to assure that solids are applied at a rate less than the solids handling capacity of the limiting layer. The limiting layer can be determined from a series of batch settling tests at various concentrations.

Kynch(26) proposed a theory for thickener operation based on the assumption that at any point in a dispersion, the settling velocity of particles is determined by the local particle density only. While Kynch's analysis created much interest(27)(28) among those involved in thickener research and design, it has been found inapplicable to flocculent materials such as activated sludge(10)(29). Fitch(29), one of the authors originally applying the Kynch analysis for design purposes has recently stated that "modern theory has not answered the unsolved problems left by Coe and Clevenger and in this respect we have not advanced much during the past half century". Based on this statement by Fitch, it would appear that most thickener design is based on Coe and Clevenger's basic theory with some modifications to facilitate collection and handling of experimental data.

BLACK RIVER WASTEWATER
TREATMENT PLANT
BALTIMORE, MARYLAND

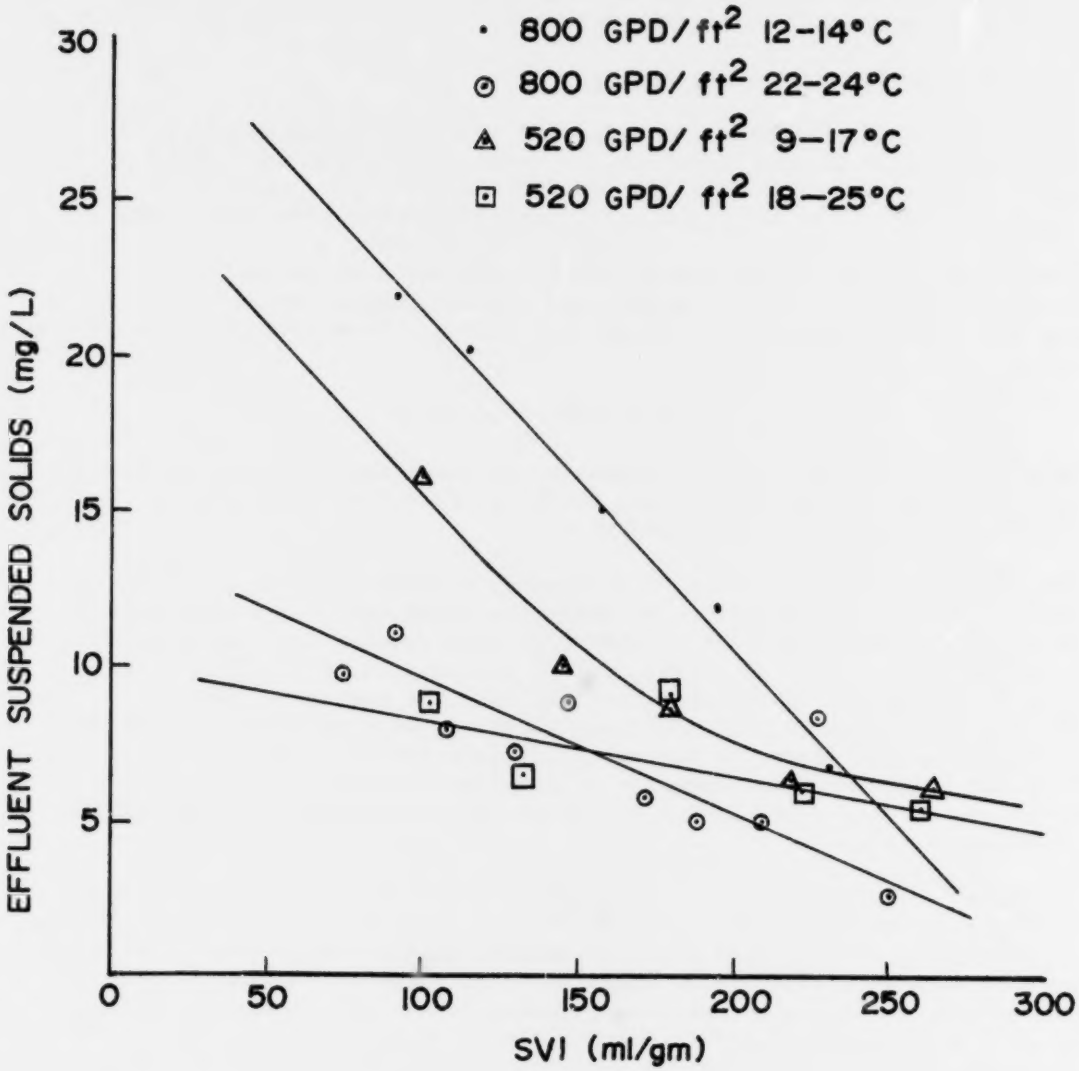


FIGURE 8

EFFECT OF SLUDGE VOLUME INDEX AND TEMPERATURE ON THE EFFLUENT SUSPENDED SOLIDS CONCENTRATION

Dick(10) found that the rate of subsidence of activated sludge was dependent on the concentration, the sludge depth and the mixing of underlying areas. He also felt that the area required to accomplish thickening in a settling tank is not fixed by the observed settling velocity of the rate-limiting concentration of sludge. It should be possible to reduce the required thickener area by adequate control of sludge depth and manipulation of the sludge to minimize the effect of interparticle forces.

Design and Operation of Final Tanks

The current basic requirements to be met in the design of final clarifiers are outlined in the Ten States Standards(30), and the Sewage Treatment Plant Design Manual of the Water Pollution Control Federation(31).

The design requirements of the Ten States Standards are summarized in Table 4. The design requirements take into account the extreme variability in flows to small plants by providing extra capacity in an attempt to level out the hydraulic surges. Allowance is also made for the various activated sludge modifications although data to substantiate this allowance is not available in the literature. No allowance is made for the various tank configurations and inlet and outlet devices that are commercially available.

The design recommendations of the Sewage Treatment Plant Design Manual are in most cases very similar if not identical to those of Ten States Standards. In addition to the usual parameters of detention time and overflow rate, the variable "solids loading" is introduced. They report that successful operation had been achieved at loads of 12 to 18 lb./sq.ft./day with sludge volume indexes under 100. As a general guideline, for mixed liquor concentration of 3000 mg/l or less with a sludge index of 100 or less and with the tank underflow at not more than 1.0% solids, the area determined by the overflow rate is adequate for the solids. When these conditions are exceeded the design area becomes a function for mass loading rather than the overflow rate.

Area and Volume Requirements

Although most regulatory agencies have defined criteria for area and volume requirements of final clarifiers, the ultimate purposes of the final clarifier must be met. That is to say, that a final sedimentation tank must provide a clarified supernatant low in suspended solids and also must concentrate the return solids to a level acceptable to return to the aeration tank.

At Sioux Falls, South Dakota, side-by-side tests were conducted in two circular tanks, 80 feet in diameter, one a center-feed and one a peripheral-feed. Effluent suspended solids concentrations of 30 mg/l were attained

TABLE 4

Design Requirements for Final Settling Tanks

Type of Process	Average Design (Flow-MGD)	Detention (Time-Hours)	Surface Settling Rates (gal/day/sq ft)
Conventional, Modified, or "High Rate" and Step Aeration	to 0.5	3.0	600
	0.5 to 1.5	2.5	700
	1.5 and up	2.0	800
Contact Stabilization	to 0.5	3.6	500
	0.5 to 1.5	3.0	600
	1.5 and up	2.5	700
Extended Aeration	to 0.05	4.0	300
	0.05 to .15	3.6	300
	.15 and up	3.0	600

General: The inlets, sludge collection and sludge withdrawal facilities shall be so designed as to minimize density currents and assure rapid return of sludge to the aeration tanks. Multiple units capable of independent operation are desirable and shall be provided in all plants where design flows exceed 0.1 MGD unless other provision is made to assure continuity of treatment.

The detention time, surface settling rate and weir overflow rate should be adjusted for the various processes to minimize the problems with sludge loadings, density currents, inlet hydraulic turbulence and occasional poor sludge settleability.

at a hydraulic loading of 1200 gpd/ft^2 for the peripheral-feed basin and 250 gpd/ft^2 for the center feed basin(1). Obviously, design loading criteria should be a function of tank geometry and inlet and outlet conditions.

In Germany, secondary basins are usually dimensioned on the basis of detention time. Detention times of from 2 to 3 hours are normally used. Pflanz(16) in reviewing the German literature reported that Schmidt-Bregas had proved, based on numerous tests, that the success of sedimentation requires hydraulic efficiency, that is, the maximum conformity of the actual with the computed detention time.

In the United States, the design of final tanks had been based on the concept of overflow rate as developed and presented by Hazen(32) and Camp(18). Recently, this concept has been questioned by Fitch(33). Fitch felt that particularly with flocculent suspensions, detention time plays a significant role and should not be discarded as a design factor. He presented data which showed that removals of some materials in an ideal basin, would be governed more by detention time than by overflow rate.

As the understanding of final clarification and the variables affecting it increase, it becomes more and more important that the design requirements be modified to reflect the differences in tank geometry, types of sludges and hydraulic regimes. It is only through modernization of the design requirements that economies in costs and improvements in performance become possible.

General Arrangements

Sedimentation basins normally are composed of four zones, an inlet zone, an outlet zone, an effective settling zone, and a solids removal zone. The location of these zones are shown schematically in Figure 9(2).

The suspension to be separated is introduced into the sedimentation tank through an inlet device of some type. The zone of turbulence extends for a distance beyond the inlet device thus rendering this zone ineffective for settling except possibly as a flocculation zone.

The area and volume of the inlet zone is a function of the inlet device. The effective settling zone is a quiescent zone in which the solids-liquid separation takes place. The clarified liquid is removed through some outlet device, generally an overflow weir. A certain area of turbulence surrounds this outlet device and is known as the outlet zone. Additional volume in the tank must be provided for the solids removal zone. The volume of this zone is a function of the suspension being clarified and the solids removal mechanism.

Katz, et al(1) report that the design of clarifiers is generally controlled by one or more of the following factors:

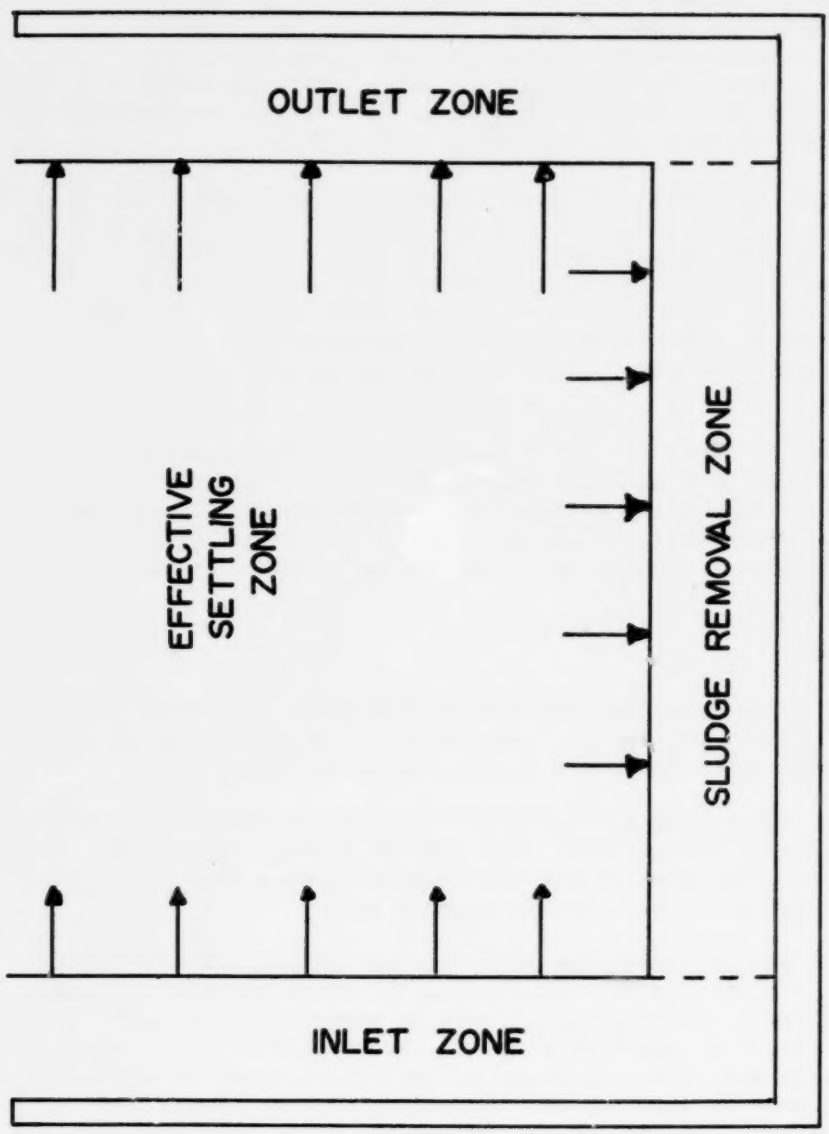


FIGURE 9 FUNCTIONAL ZONES OF A FINAL SETTLING TANK

1. Conditions such as short-circuiting, turbulence, density currents, and inlet and outlet conditions will affect all of the following factors.

2. The method of sludge withdrawal must be considered in light of the specific application.

3. The area required for clarification is related to the volumetric overflow rate. The vertical liquid rise rate at any level must be less than the solids subsidence rate. The area and volume required to produce the desired underflow solids concentration may be an important design criterion. The allowable solids detention time in the basin is dependent on its biological properties.

Each of the previous criteria will be discussed according to how it affects sedimentation tank design and operation.

HYDRAULIC FACTORS

Inlet Zone

The purpose of an inlet device is to uniformly distribute the flow across the cross-sectional area of flow of the tank. Ingersoll(34) has stated that his results, which substantiate the claims for most other investigators, confirm that inlet conditions are far more important than those at the outlet.

A basin inlet much accomplish both horizontal and vertical distribution of the flow over the entire cross-sectional area of the tank in order to effectively utilize the entire tank volume for sedimentation. Hydraulic equality is obtained by either subjecting the dividing flow to equal frictional resistances or by inserting at each point of discharge a controlling headloss that is large in comparison to the resistance between inlets(20).

Numerous types of inlet devices are reported in the literature(35)(36). Since each inlet device must be integrated with the configuration of the sedimentation basin, the overall distribution efficiency will depend on the correctness of this integration. Double vertical slotted inlet baffles have been reported to be quite successful in rectangular basins.

Circular sedimentation basins are designed with both center and peripheral inlets. An inlet scheme for each type is shown in Figures 10 and 11.

The circular, center feed basin disperses flow through an inlet device consisting of a number of baffles to dissipate the kinetic energy. Withdrawal of the clarified liquid is at the periphery.

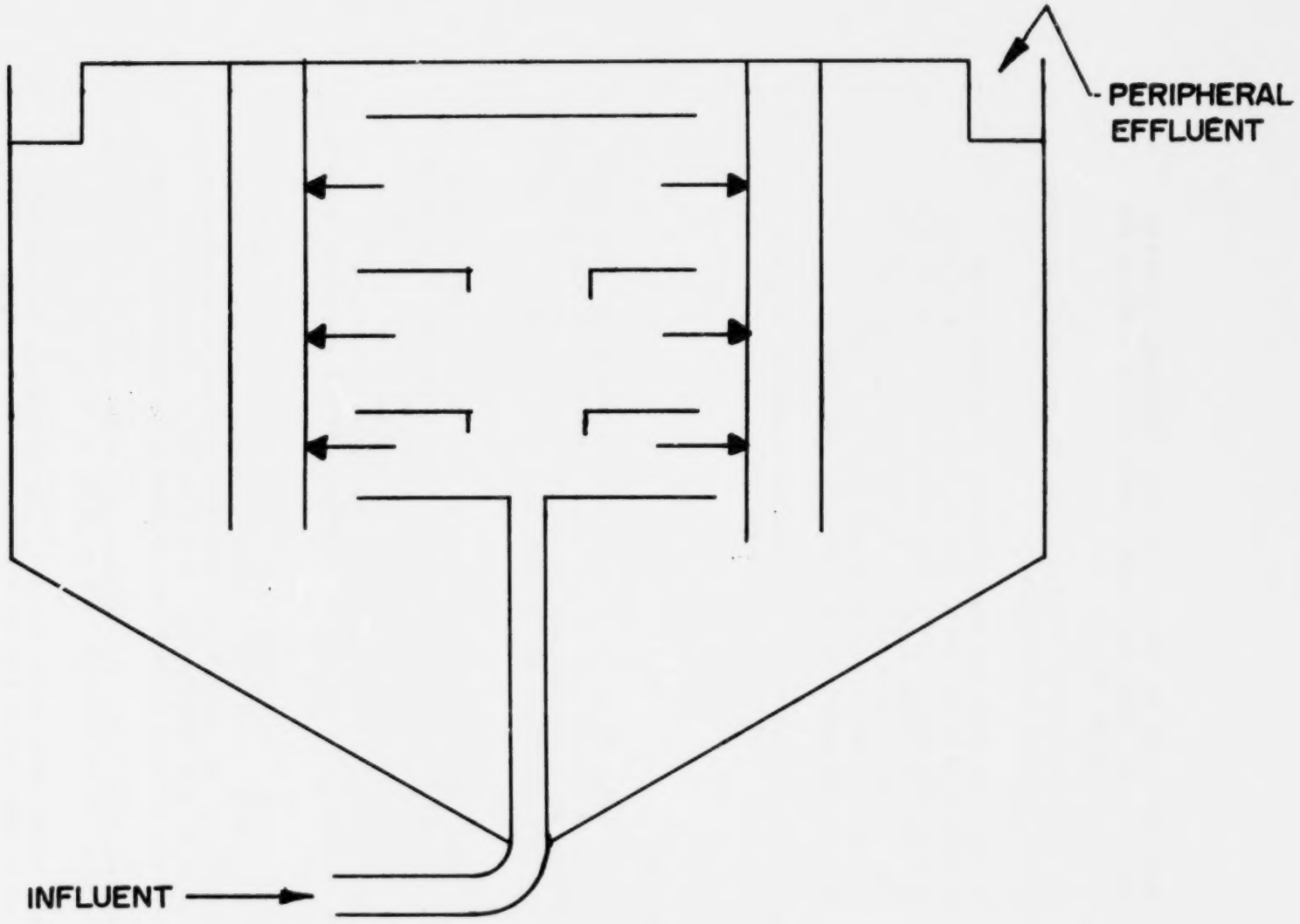


FIGURE 10
CENTER FEED BASIN

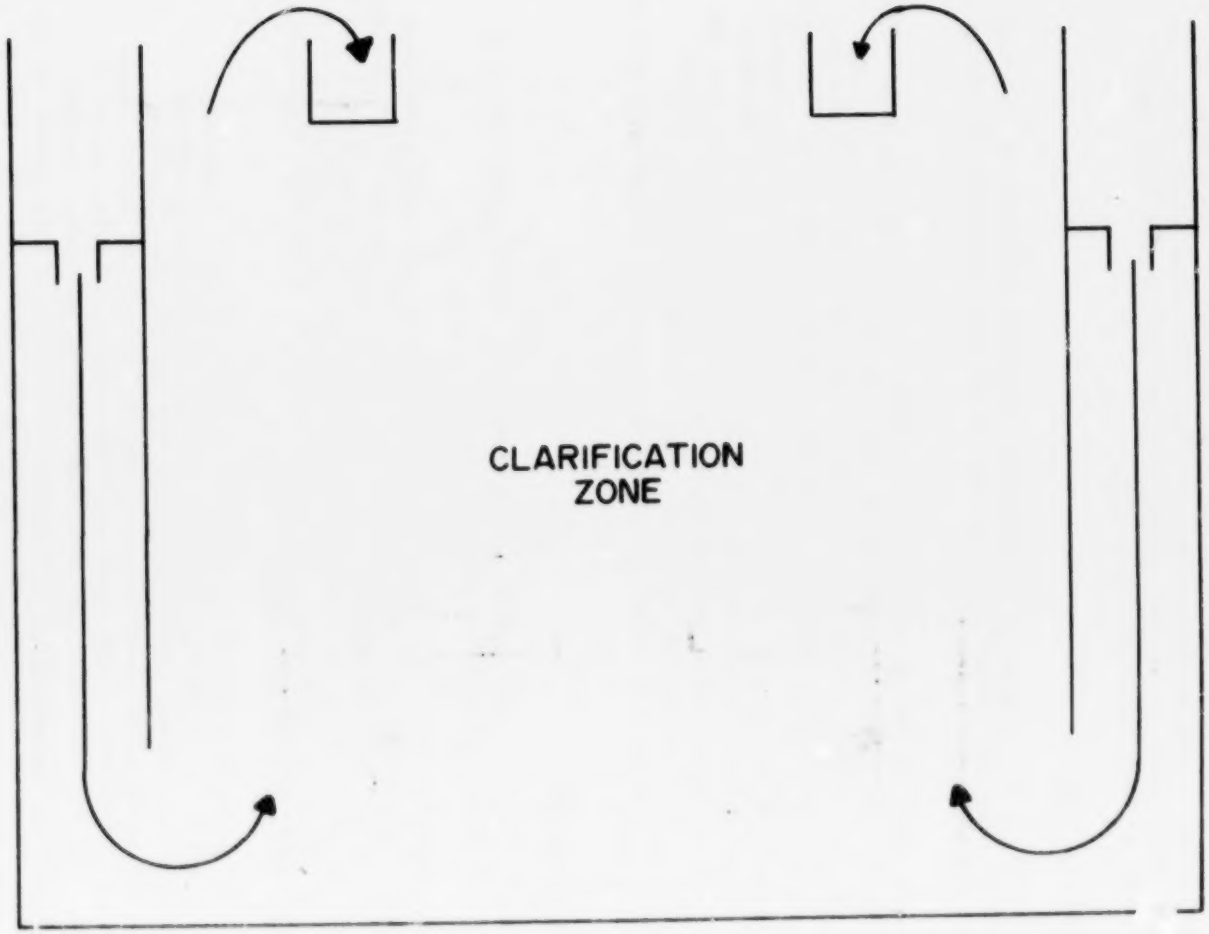


FIGURE II

PERIPHERAL FEED BASIN

A peripheral feed tank is shown in Figure 11. Flow is introduced around the periphery of the tank and below a skirt baffle. A peripheral inlet device such as this is advantageous because it permits a large distribution area, thus minimizing the velocity gradients at the point of introduction to the clarification zone. A comparative study of the hydraulic characteristics of center and peripheral feed basins has shown the peripheral feed basin to be more effective than the center feed basin particularly at high overflow rates(37).

Outlet Zone

Outlet devices are designed to collect the effluent uniformly at the outlet with minimal take-off velocities required to prevent carryover of sludge solids to the effluent channel.

The commonly used method of effluent collection is the overflow weir. The procedures for designing effluent weirs are contained in most sanitary engineering textbooks. Allowable weir rates vary with the configuration of the sedimentation tank and with different regulatory agencies. Anderson (38) gave a maximum of 20,000 gallons per day per foot of weir for weirs located away from the upturn of the density current. For weirs located within the upturn zone, the rate should not exceed 15,000 gallons per day per foot. These values were determined for circular tanks with center-feed. Pflanz(16) reported on three secondary sedimentation tanks in Germany with weir overflow rates ranging from 10,700 to 46,600 gals per day per foot of weir.

Settling Zone

Since clarification or separation of the suspended solids takes place in this zone, quiescent conditions should exist.

Particular emphasis should be placed on the design of the cross-sectional area so that the horizontal velocity in this zone will not be large enough to scour solids which have already been deposited. Ingersoll, McKee, and Brooks(39) recommend that the ratio of the critical tank displacement velocity to the settling velocity of the critical size particle should be less than 9 to 15 to prevent scour.

Anderson(38) reported density currents in final clarifiers treating activated sludge. He attributed these currents to the difference in density between the mixed liquor suspended solids and the clarified liquid in the tank. This difference in density caused the mixed liquor to plunge to the bottom of the tank and flow along the bottom until some obstruction was encountered. The encounter with an obstacle, usually the side of the tank, induces a counter-current in the upper levels of the tank. These currents are shown schematically in Figure 12. No density currents were observed in primary settling tanks. Gould(40) reported similar density currents at the New York City Sewage Treatment plants.

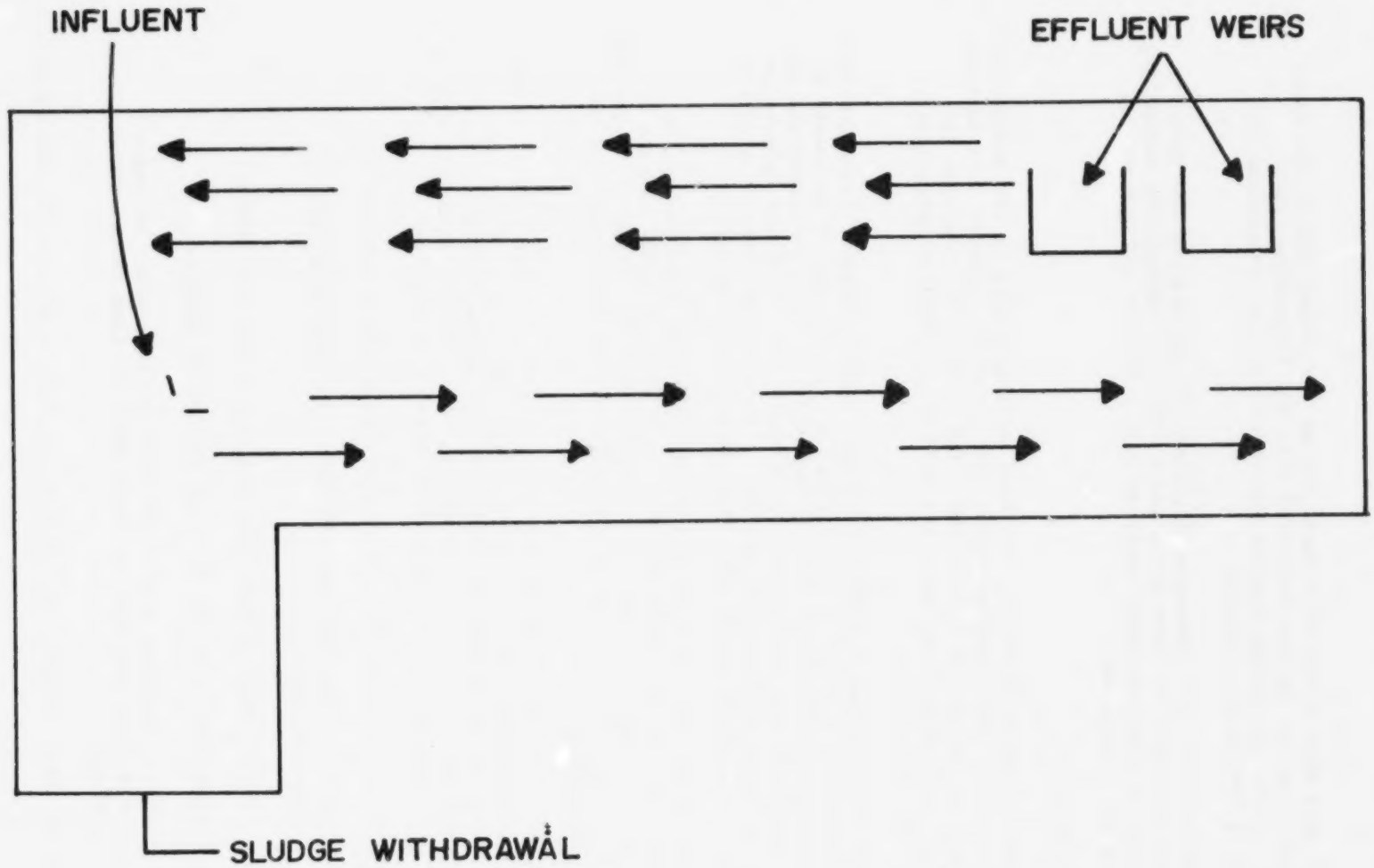


FIGURE 12 DENSITY CURRENTS IN A FINAL CLARIFIER

He found that when sludge withdrawal was at the outlet end of the tank, the velocity of the current decreased with the increasing sludge density. Thus, the sludge blanket was successful in diminishing the velocity of the density current.

Fitch and Lutz(41) have presented a method for calculating the velocity of density currents and have discussed a number of methods for minimizing the effects of these currents based on both hydraulic and hydrostatic stabilization techniques.

SHORT CIRCUITING

The subject of short-circuiting has been much discussed in the sedimentation literature. Efficient sedimentation procedure dictates that maximum use be made of the entire tank volume. Dye studies to determine the effects of short circuiting have been made by a number of investigators (37)(38)(42)(43)(44)(45).

Early tracer studies were performed using a sodium chloride technique(46). This method has been shown to cause density currents and is no longer a generally accepted procedure(34). Modern tracers include Fluorescein Dye and Rubidium^{86}(43) and radioactive potassium^{42}(44). Katz and Geinopolos (47) used a water insoluble oil Red T.A.X. to study the retention characteristics of activated sludge particles in a final clarifier.

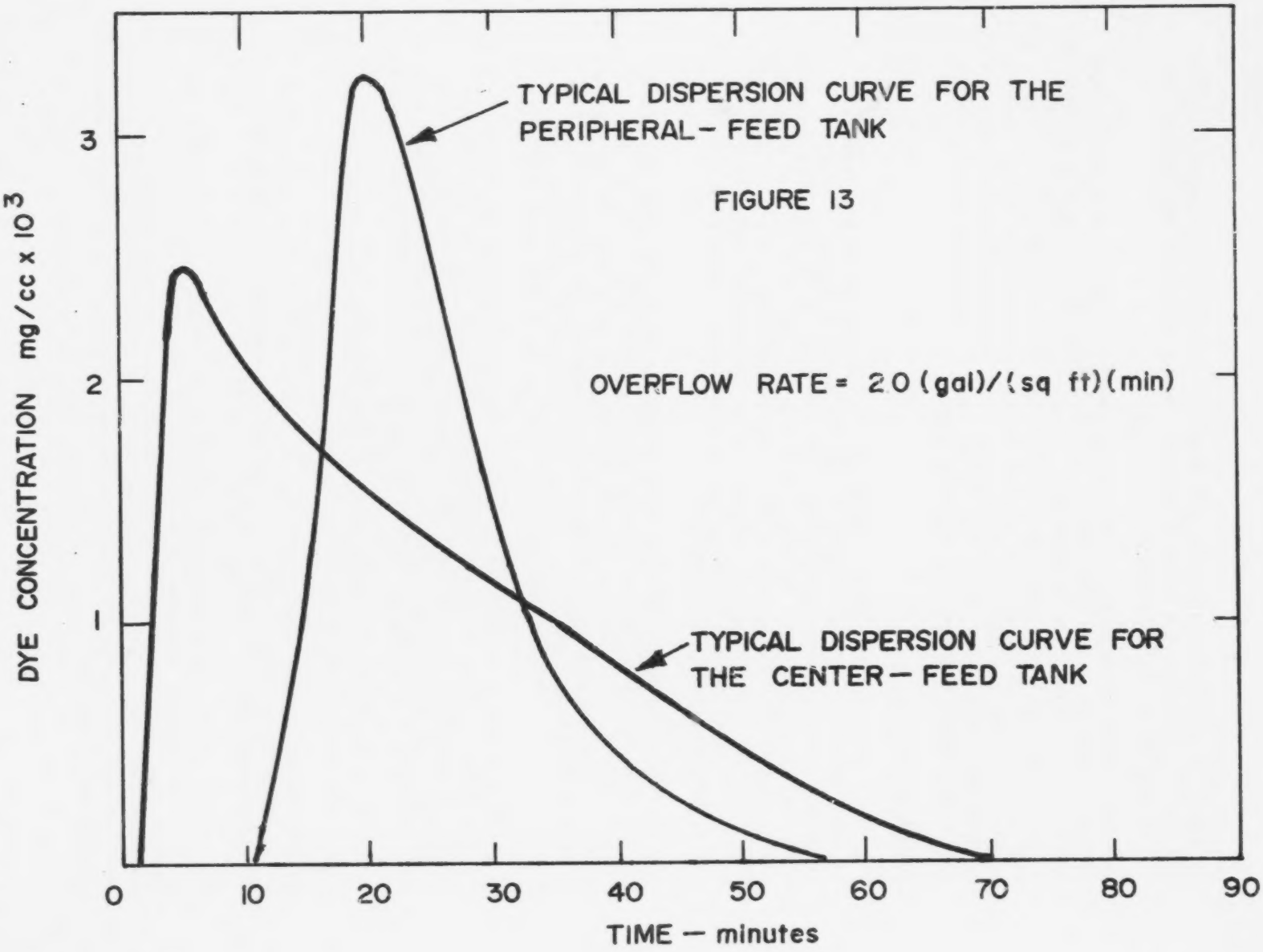
The general procedure for hydraulic studies involves introducing the tracer material into the basin and then collecting effluent samples for a period of time and analyzing for dye concentration. Figure 13 shows the results of a typical dye study on the hydraulic characteristics of two circular basins(37).

The dispersion curve was then analyzed to determine T_i, the time at which the initial appearance of dye occurred; T_{max}, the time at which the maximum concentration of dye was observed; T_f, the most probable flow-through time; and T_{90}/T_{10}, the dispersion index. These parameters were then used to characterize the hydraulic behavior of the tank.

Fair and Geyer(20) report that short-circuiting may be used by:

1. Eddy currents that are set up by the inertia of the incoming flow.
2. Wind induced currents when the basins are not covered.
3. Convection currents that are thermal in origin.
4. Density currents that cause cold or heavy water to underrun a basin and warm or light water to flow across its surface.

Any or all of these factors can cause the departure of a basin from ideal.


TYPICAL DISPERSION CURVE FOR THE PERIPHERAL- FEED TANK
FIGURE 13
OVERFLOW RATE = 2.0 (gal)/(sq ft)(min)
TYPICAL DISPERSION CURVE FOR THE CENTER- FEED TANK
DYE CONCENTRATION mg/cc x 10^3
TIME — minutes
0
10
20
30
40
50
60
70
80
90
1
2
3

It would be difficult to accurately quantify the effect of short-circuiting but the importance of this parameter can be judged by the efforts of design engineers to minimize it.

Camp(48), while stressing the concept of overflow rate as the controlling factor in sedimentation tank design, also pointed out that short-circuiting can seriously affect the performance. Eliassen(49) questioned whether short-circuiting really would deteriorate the performance since short-circuiting could occur without changes in surface overflow rate. Camp pointed out that the overflow rate could also be defined as depth divided by detention time which shows that a decrease in detention time by short-circuiting would increase the overflow rate.

SLUDGE WITHDRAWAL

The rapid removal of settled sludge from the bottom of final clarifiers is very important to the overall sewage treatment process. Shapiro, et al(50) have shown that phosphates removed from solution during aeration would be released back to solution under conditions of low redox potential. They proposed rapid removal of solids from the settling basin as a method of preventing this phosphate release. With the present emphasis in the pollution control field on phosphate removal, rapid removal of settled sludge becomes even more important.

A number of authors(51)(52)(53), have commented on the problem of rising sludge in final clarifiers. This problem has been attributed to denitrification with the subsequent release of nitrogen gas. The rising gas bubbles cause quantities of sludge to be buoyed to the surface with a decrease in effluent quality. Sawyer and Bradney(51) found the best solution to thi problem was the rapid removal of sludge from the tank.

Although prolonged anaerobisis(24 hours) has little effect on the assimilative capacity of an activated sludge and, in fact, has an inhibitory effect on filamentous organisms(4), it would appear that the benefits to be gained from rapid sludge removal from the final clarifier dictate that solids should be removed as rapidly as is feasible from the final clarifier.

There are essentially two methods for removing solids from final clarifiers, mechanical and hydraulic collectors.

Mechanical collectors move the sludge to a centrally located collection point by means of plows, rakes, or flights. A controversy exists as to the exact mechanism of sludge movement(38)(54).

Regardless of the exact mechanism of movement, it has been shown that the efficiency of sludge removal is a function of the differential velocity between the flight speed and the mean basin flowthrough velocity(1). Additionally, when the sludge movement is in the same direction as the flow through the basin, the sludge carrying capacity of the flights was significantly approved.

Hydraulic sludge collectors remove sludge from the point of deposit rather than conveying them to a central collection point as for mechanical collectors. Hydraulic sludge collectors are well adapted to activated sludge because of their ability to rapidly remove sludge from the point of deposition.

DEPOSITION PATTERNS

The deposition pattern of solids in final clarifiers treating activated sludge have been reported by Anderson(38), Pflanz(16), and Albrecht et al(55).

Anderson(38) studied a center-feed peripheral-drawoff circular basin with a 126 foot diameter. He made soundings on a radius of the tank and found suspended solids distributions ranging from 1 ppm at the surface to 18,900 ppm in the sludge drawoff hopper. The solids profiles were slightly raised near the effluent weirs indicating the effect of velocity currents near the overflow weir. Velocity measurements are also shown and indicate the beneficial effect of the sludge blanket in reducing the sludge density currents. Although the suspended solids concentration in the sludge blanket immediately above the sludge hopper was 10,000 ppm, its depth was only about two feet. High sludge drawoff rates could draw quantities of water with significantly less solids concentration. These low solids levels are less than the MLSS concentration and over a period of time would materially reduce the activated solids in the aeration tank.

Pflanz(16) presented an excellent set of data showing the change in solids profile for corresponding changes in influent flow rate. As the flow rate increased from $15m^3/hr$ to $80m^3/hr$, the suspended solids concentration at the effluent weir rose from 3 to 40 mg/l. Meanwhile, the return sludge concentration increased from 8.4 gms/l to 21.6 gms/l. This increase in return sludge concentration shows the benefit of maintaining a deep sludge blanket over the sludge drawoff hopper. However, the benefits gained by increased concentration of return sludge were partially offset by the deterioration in effluent quality.

For the basin studied by Pflanz, it is interesting to note the solids concentration at the floor in the outer portions of the lightly loaded basin. Since there are very few solids in the outer one-half of the basin, a hydraulic sludge collector would be drawing nearly clear water from these portions of the basin. At the higher loading rate, the entire basin, except for the outermost portion, contained a sludge blanket of some depth.

Albrecht et al(55) found that the solids deposition pattern in the immediate vicinity of the inlet is a function of the inlet well design. They also compared two activated sludges, one a healthy aerobic sludge, the other sludge being deficient in oxygen. The aerobic sludge exhibited a sharp sludge blanket interface while the oxygen deficient sludge exhibited a relatively constant sludge concentration in the top portion of the tank. In this case the environmental conditions in the aeration

tank appeared to be the controlling factor in the settling tank.

SUMMARY

This literature search has attempted to report the commonly accepted procedures for the design and operation of final clarifiers. Current design requirements were summarized by the Ten States Standards(30) and the WPCF Sewage Treatment Design Manual(31). Based on the results of this survey of the literature, it may be concluded that the design requirements are general guidelines obtained through years of practical experience. They do not include considerations for final clarifier geometry or hydraulic effectiveness.

The variables discussed in the literature review which were indicated as being important to either the clarification or thickening functions of a final clarifier are summarized in Table 5. The main effects and interacting effects of these variables were analyzed in an attempt to arrive at a dimensionally balanced equation. This equation will be presented in the Results Section.

The techniques of multiple regression will also be used in the formulation of the mathematical model.

TABLE 5

Variables Affecting the Clarification and Thickening of Activated Sludge

Clarification

A. Tank Characteristics
   1. Surface area
   2. Depth
   3. Weir length and position
   4. Inlet device
   5. Hydraulic efficiency

B. Sludge Characteristics
   1. Settling rate
   2. Compaction characteristics (SVI)
   3. MLSS

C. Operational Characteristics
   1. Overflow rate
   2. Detention time
   3. Weir overflow rate
   4. Mass loading
   5. Mixed liquor flow

D. Biological Characteristics
   1. Activate sludge mode
   2. BOD load g

Thickening

A. Tank Characteristics
   1. Surface area
   2. Tank depth
   3. Type of sludge removal mechanism

B. Sludge Characteristics
   1. Settling rate
   2. Compaction characteristics (SVI)
   3. MLSS

C. Operational Characteristics
   1. Mass loading
   2. Return rate
   3. Sludge blanket depth
   4. Mixed liquor flow

THEORETICAL DEVELOPMENT

The performance of a final clarifier is measured by the quality of the effluent in terms of suspended solids and by the concentration of the return sludge. The tank must provide a high degree of clarification and at the same time, it must be capable of providing a return sludge of reasonable concentration. An economic design must incorporate both the clarification and the thickening functions of a final clarifier.

The most commonly recommended procedures for clarifier and thickener designs involve the selection of a unit area for each of the functions and then selecting the larger of the two unit areas as the area controlling variable.

Area for Sedimentation

The procedure for determining the unit area required for sedimentation involves the measurement of the settling rate of the sludge in question, using the laboratory settling rate procedure outlined in the Experimental Procedure Section. The settling rate expressed in feet per hour can be converted to an overflow rate by means of the following equation:

$$OR = (SR \text{ ft/hr}) \times 7.45 \text{ gal/ft}^3 \times 24 \text{ hr/day} \qquad (1)$$

where, OR = overflow rate (gallons/day/ft^2)

SR = settling rate (ft/hr)

This overflow rate based on the laboratory settling rate thus determines the maximum overflow rate which can be expected to produce a reasonably clarified effluent. Hydraulic considerations and short-circuiting introduce inefficiencies in the final clarifier which should be accounted for in the design. An adjusted overflow rate can be calculated which will make allowances for the above mentioned inefficiencies. This relationship is given by Equation 2.

$$ORD = K_1 \times OR \qquad (2)$$

where, ORD = design overflow rate (gpd/ft^2)

OR = laboratory overflow rate (gpd/ft^2)

K_1 = a dimensionless correction factor

The magnitude of the constant K_1 is a function of the tank configuration, the inlet and outlet design, and the hydraulic efficiency of the clarifiers. Although exact measurements of K_1 are not available, the magnitude of this factor ranges from about 0.5 to 0.8 for the various final settlers which are commercially available. Determination of these K_1 values is an area requiring future research.

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It should be pointed out that the determination of the laboratory overflow rate, OR, should be made at various mixed liquor suspended solids concentrations. The range of concentrations should include the maximum mixed liquor suspended solids concentration that might be expected since this will impose the most severe solids separation condition on the final clarifier. The laboratory overflow rate is very definitely a function of mixed liquor suspended solids concentration. However, this function has not been found to be consistent for activated sludges from different plants(10). The development of an equation relating mixed liquor suspended solids to the laboratory overflow rate would provide a significant addition to the theory and practice of sedimentation since it would allow the estimation of the settling rates of various sludges without performing laboratory testing. This would be particularly useful in simulation models where laboratory data are unavailable but the subsidence rate of the sludge is desirable.

Area for Thickening

The area required for thickening is often not considered in the design of final clarifiers.

This factor is, however, very important particularly when the clarifier is expected to handle high (>5,000 mg/l) mixed liquor suspended solids concentrations or when sludges with poor subsidence characteristics are encountered.

Dick(56) has proposed that in a continuous thickener, solids are transmitted downward by two mechanisms:

1. By subsidence under the influence of gravity. This sedimentation occurs at a velocity V_i which in turn is primarily a function of the initial solids concentration C_i.

2. By bulk transport as a result of sludge removal. This occurs at a velocity μ which depends on the rate at which solids are removed from the bottom of the basin.

The total possible flux, S, of solids through a layer with solids concentration C_i is given by:

$$S = \underbrace{C_i V_i}_{\text{Subsidence}} + \underbrace{C_i \cdot \mu}_{\text{Sludge Withdrawal}} \qquad (3)$$

Associated with the settling velocity - concentration relationship there exists a limiting solids handling capacity S_L which determines the area required for thickening.

By utilizing a curve relating the settling velocity V_i to the mixed liquor suspended solids concentration C_o and the relationship given in Equation (3), a plot can be made of the return sludge concentration versus the solid flux. This curve will then indicate the limiting solids flux rate S_L and will enable the calculation of the area required for thickening.

$$A = \frac{Q_o \cdot C_o \cdot 8.34}{S_L} \qquad (4)$$

Where:

A = area required for thickening (ft^2)

Q_o = mixed liquor flow rate (mgd)

C_o = mixed liquor solids concentration (mg/l)

This approach, although easy to use, is not functional in a simulation model since no a priori knowledge of the MLSS settling rate relationship is available. As with the calculation for area requirements for sedimentation, the lack of knowledge of a consistent relationship to describe the settling rate of a sludge as a function of its concentration makes a theoretical analysis of thickening unavailable for use in a simulation model.

EXPERIMENTAL PROCEDURES

Experimental Approach

The original work scope for this project required the development of a mathematical model to predict the performance of a single final clarifier. This was later expanded in an attempt to develop a more generally applicable series of equations which could be used to predict the performance of final settlers in general.

As a part of the original work scope, a detailed experimental program was conducted at the Racine, Wisconsin Water Pollution Control Plant. This program involved the testing of two peripheral feed final clarifiers under a variety of hydraulic and operational conditions.

An experimental program was designed using the concepts of Box-Wilson(57). This technique allows the simultaneous variation of the parameters under consideration. The technique is well suited to this type of study since the formulation of empirical equations with multiple regression techniques requires a balanced spread of the variables under consideration.

The experimental design involved testing of three variables, mixed liquor suspended solids, sludge blanket depth and overflow rate at five different levels with replicates at the central point. Analysis of the data at the conclusion of this design showed that the dependent variable, effluent suspended solids, varied over only a small range despite wide changes in the independent variables. Additional data were collected and combined with those collected in the experimental design to broaden the results of the study.

In order to expand the usefulness of the model, data were also collected at Brookfield and Fort Atkinson, Wisconsin Sewage Treatment Plants on rectangular final clarifiers. At these plants, close control of the flows was not possible. However, the data collected at these plants did incorporate into the model the variability in sludges which can be found at various plants.

Additional data were also obtained from operating records at various plants throughout the country. Of particular use was the data from the Hyperion, California plant supplied by Mr. Robert Smith, Chief, Operations Research Section of FWPCA.

Description of Plants Studied

During the course of this project, studies were made at three sewage treatment plants in the southern Wisconsin area in order to obtain data under controlled experimental conditions for use in the construction and testing of the mathematical model.

A description of the three plants at which data were collected as well as the design data from those plants from which operating data were used follows.

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Racine, Wisconsin Sewage Treatment Plant

The majority of the test work for this project was performed at the Racine, Wisconsin, Water Pollution Control Plant. This plant accepts the domestic and industrial wastes of the City of Racine as well as the towns of Mount Pleasant and Caledonia, and the villages of North Bay, Lakeside, and Colonial Heights.

The plant is designed to handle 23 million gallons per day (mgd) of raw waste for primary treatment and 12 mgd for secondary treatment. A flow diagram for the plant is shown in Figure 14. The basic plant design criteria are presented in Tables 6 and 7.

During the period of the test program, the plant was operating utilizing two different activated sludge modifications. Prior to April 15th, 1969, the plant was operated as Contact Stabilization and thereafter as the Kraus Process.

The waste at the Racine Plant is a mixture of both industrial and domestic sewages and is low strength based both on BOD and suspended solids. For the period of this study, the average raw waste BOD and suspended solids were 102 and 142 mg/l respectively. The per capita contribution of flow was 227 gallons per day.

Because of an overloaded primary sedimentation system, the average removal of suspended solids and BOD in the primary settling tanks was approximately 20 and 10% respectively. The primary system does, however, perform the useful function of removing grease and other floatable materials.

The activated sludge tanks are of a conventional design, being 15 feet deep by 30 feet wide by 168 feet long. The flow pattern is of the down, around, and back type giving an effective length of 336 feet for the Contact Stabilization and Kraus Processes. Aeration is provided by a Kraus dual aeration system with air being introduced at high and low levels on opposite sides of the tank. Periodic checks of dissolved oxygen at different points along the aeration tank indicated that dissolved oxygen levels greater than 2.0 mg/l existed in all cases.

Final clarification of the activated sludge is accomplished in two peripheral-feed, center-takeoff clarifiers equipped with hydraulic sludge collectors. These final clarifiers are 85 feet in diameter with a side wall depth of 12.0 feet. The Tow-Bro® is equipped with two headers and the time for one revolution is 21 minutes. This means that each position in the tank is being swept once every 10.5 minutes.

The return sludge pumps were a combination of both fixed and variable speed and were valved in such a manner that nearly any return sludge flow was possible.

A schematic diagram of the return sludge piping arrangement is shown in Figure 15. Return sludge sampling valves were installed in each pump head and return sludge samples were withdrawn at that point.

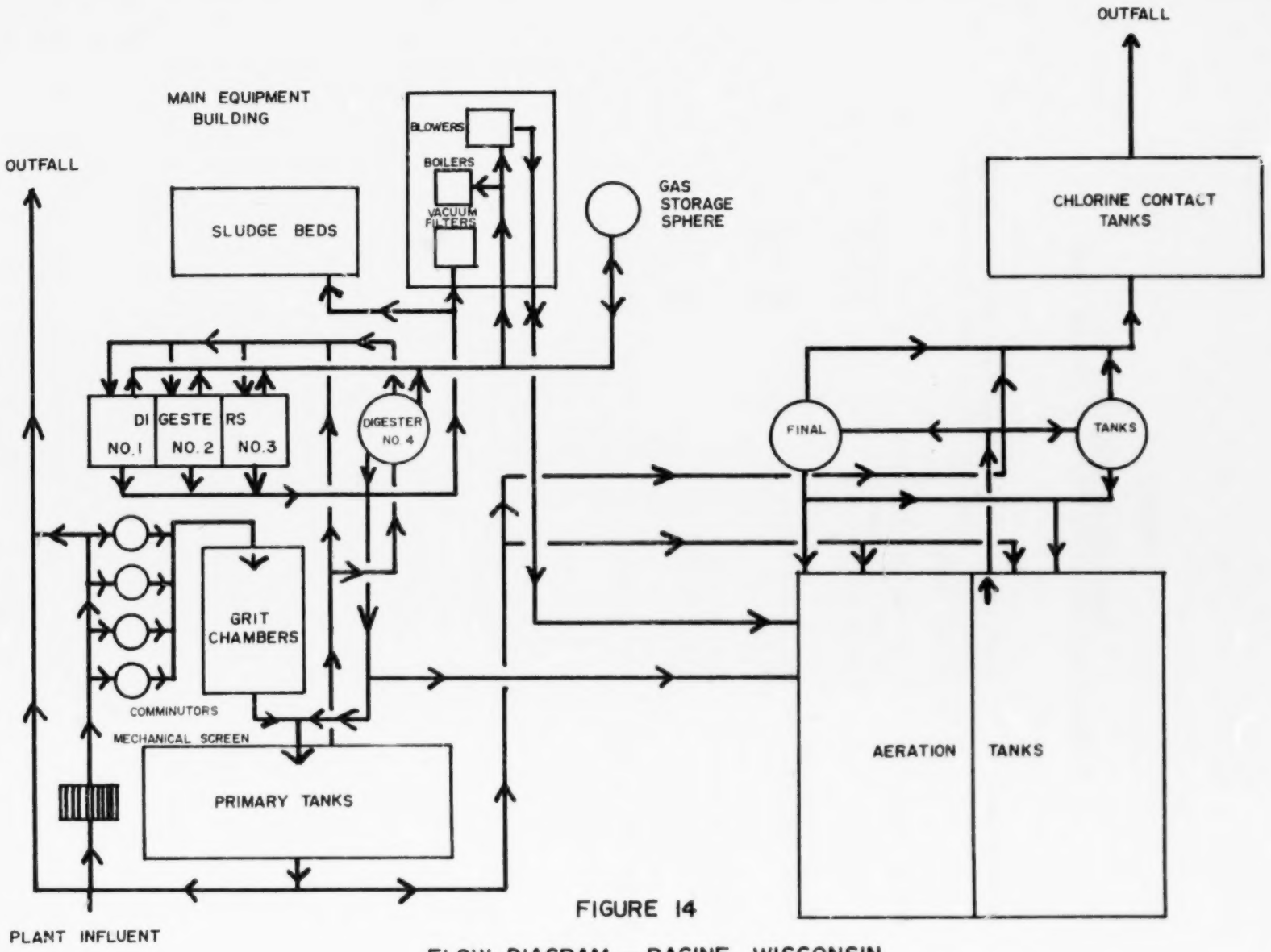


FIGURE 14

FLOW DIAGRAM — RACINE, WISCONSIN
WATER POLLUTION CONTROL PLANT

TABLE 6

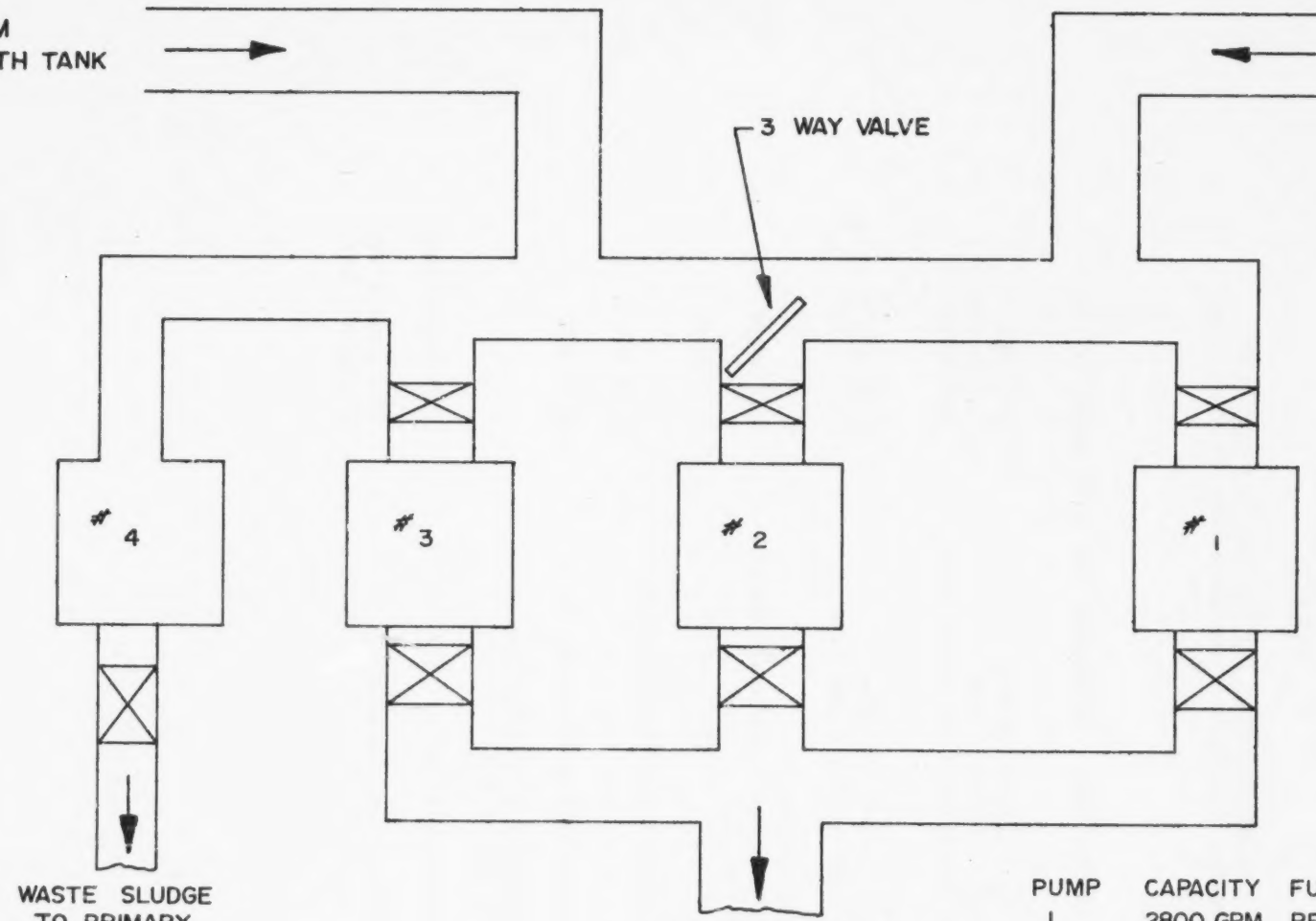
Basic Design Criteria
Racine, Wisconsin Water Pollution Control Plant

Description:	Modified activated sludge with separate sludge digestion, sludge filtering, and chlorination of effluent.	
Design Flows:	23.0 MGD for Year 1974	
	Primary Treatment	23.0 MGD
	Secondary Effluent	12.0 MGD
Design Wastewater Characteristics:	Suspended Solids	200 mg/l
	BOD_5	200 mg/l
Expected Overall Removals:	Design Flow	
	Suspended Solids	81.0 percent
	BOD_5	67.5 percent
Design Population:	Connected 1974	125,000

TABLE 7

Design Criteria for Various Process Units
Racine, Wisconsin Water Pollution Control Plant

Process Units:	
Primary Settling Tank - Detention Time 1.56 hours @23 MGD	
Aeration Tanks	
Contact Stabilization Process	12.0 MGD
Aeration Detention Time	1.57 hours
Sludge Reaeration Detention Time	5.2 hours
Kraus Process	
Aeration Detention Time	1.58 hours
Nitrification Detention Time	23.9 hours
Conventional Process	
Detention Time - Raw Waste Basis	6.0 hours
Detention Time - Mixed Liquor Basis (50% return)	4.0 hours
Final Settling Tanks Detention Time - @12.0 MGD	2.04 hours
Return Sludge Pumping Capacity	12.0 MGD
Chlorine Contract Detention Time - 15.0 minutes @70.0 MGD	--
Digester Volume per Capita	2.35 cubic feet
Sludge Vacuum Filter Capacity	3000 pounds dry solids per hour



PUMP	CAPACITY	FUNCTION
1	2800 GPM	RETURN
2	2800 GPM	RETURN
3	1130 GPM	RETURN
4	700 GPM	WASTE

FIGURE 15

PIPING DIAGRAM FOR RETURN SLUDGE GALLERY

Return sludge flows were metered by means of Fischer and Porter magnetic flow meters installed in the return sludge lines.

The effluent flows from the final clarifiers were measured in two Parshall flumes and transmitted to two readout boxes and a strip chart recorder.

Brookfield Sewage Treatment Plant

The Brookfield Sewage Treatment Plant serves the City of Brookfield, Wisconsin, with a population of approximately 11,000 people. The plant was originally designed as Conventional Activated Sludge with a capacity of 1 mgd.

The average BOD and suspended solids of the raw sewage are 100 and 150 mg/l respectively. The average daily flow through the plant is approximately 1.5 mgd.

A flow diagram together with sizes for the treatment units important to this study is shown in Figure 16.

Average BOD and suspended solids reductions through the primary settling tanks are 35 and 60%, respectively. This results in a light loading on the activated sludge system.

As a result of problems with high SVI's the operator has recently changed to a modified Kraus activated sludge system with a resultant improvement in sludge settleability.

The final clarifiers consist of two rectangular final settling tanks 60 by 14 by 10.2 feet deep. Both tanks are equipped with mechanical sludge scrapers with sludge being scraped to the inlet end. There are 56 linear feet of effluent weir in each tank located approximately 2.5 feet from the end of the tank. The average mixed liquor detention time in the final clarifier is 1.5 hours.

Fort Atkinson Sewage Treatment Plant

The treatment plant serving the City of Fort Atkinson was constructed in 1934 as a conventional activated sludge plant, with a capacity of 0.5 mgd. In 1960, extensive additions, including a trickling filter, enlarged aeration and final clarifier tanks, and the additional digesters increased the plant capacity to 1.5 mgd.

A flow diagram together with sizes for the treatment units are shown in Figure 17.

The activated sludge system is used to treat the effluent from the roughing filter. The effluent BOD of the roughing filter ranged from 18 to 60 mg/l. The filter effluent also contained the sloughed bacterial slimes from the filter. Incorporation of this slime into the activated sludge mixed liquors resulted in a sludge with excellent subsidence characteristics.

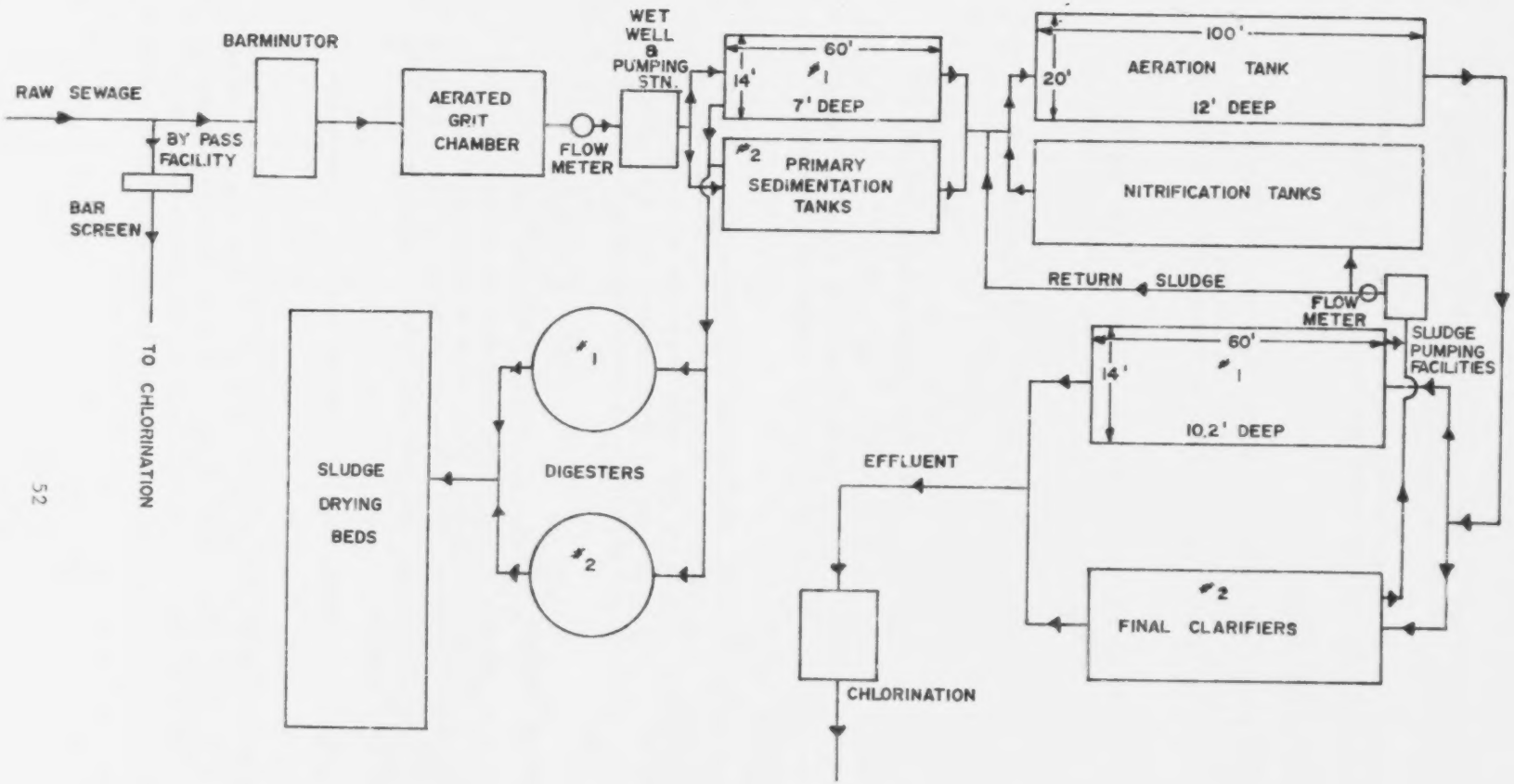


FIGURE 16

FLOW DIAGRAM – BROOKFIELD SEWAGE TREATMENT PLANT

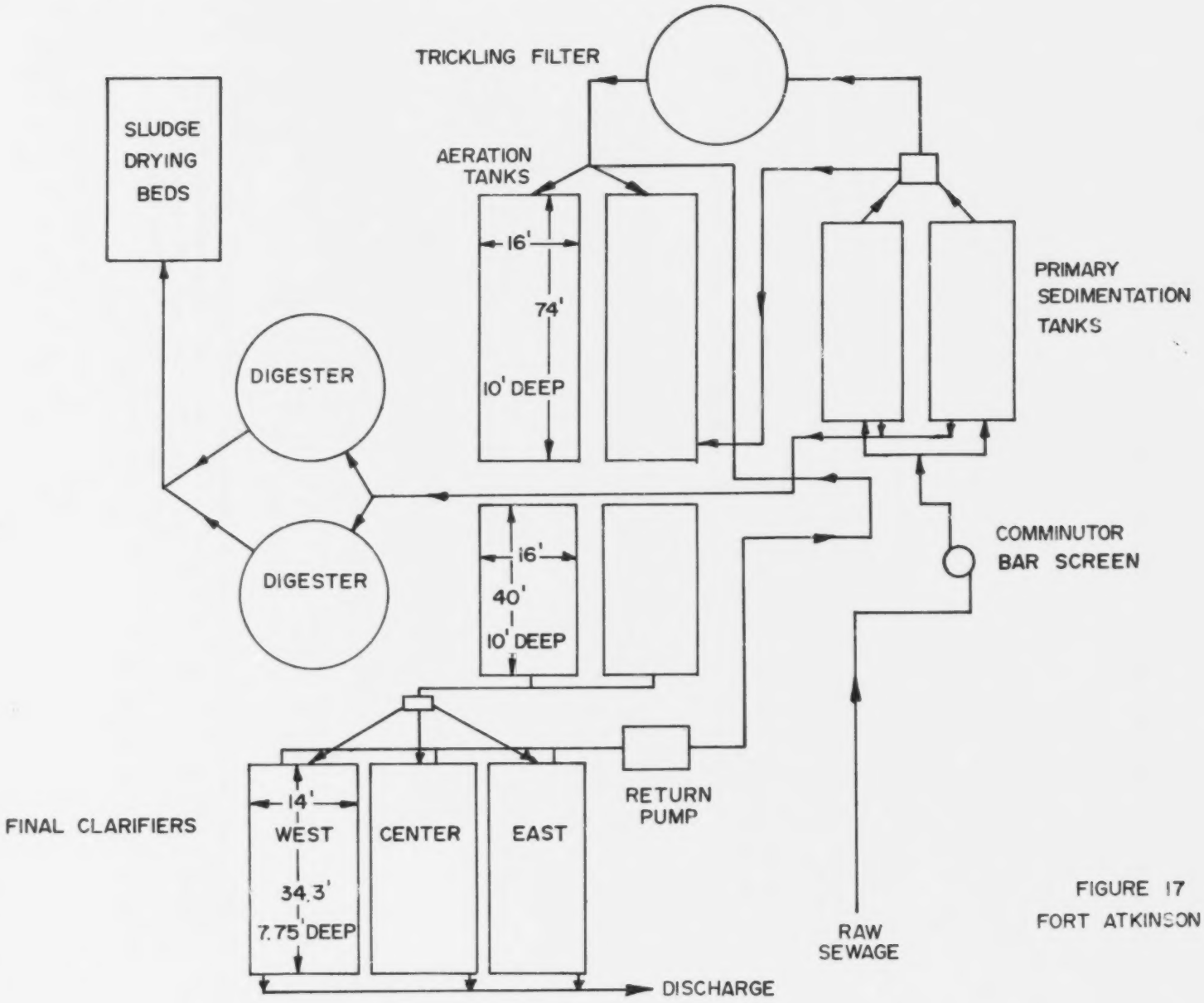


FIGURE 17
FORT ATKINSON

The final clarifiers of the plant consist of three rectangular tanks each 14 feet wide, 34.3 feet long and 7.75 feet deep. The weir length per tank is 47 feet. All three tanks are equipped with mechanical sludge scrapers. Sludge is scraped to the inlet end. Return sludge flow is adjusted with telescopic valves and a constant speed pump. The average mixed liquor detention time in the final clarifier is .75 hours.

ANALYTICAL PROCEDURES AND EXPERIMENTAL ERROR

Suspended Solids

Suspended solids were determined by the membrane filter technique as described by Engelbrecht and McKinney(58). Gelman Instrument Company glass fiber filters, Type A, 47 mm diameter were used. Gelman reports a 99.7% retention efficiency using the DOP test (0.3μ). A detailed description of the preparation and handling of the filters follows.

Preparation of Filters

The filters were washed with 100 ml of distilled water prior to use. This procedure was followed in order to purge from the filters any fines which might be washed from the filter during filtration of the sample. After washing, the filters were placed in aluminum pans and dried at 103°C for a minimum of four hours. The aluminum pan and filter were then cooled for one hour in a dessicator and weighed. The tared aluminum pans and filters were stored in aluminum cake pans for transportation to the test site.

The suspended solids filtrations were performed at the field test site using a standard Millipore Filter apparatus and a vacuum pump. Aliquot size was variable depending on the suspended solids concentration but in general aliquots were as large as possible. Typical aliquot sizes are shown in Table 8.

TABLE 8

Typical Aliquot Size for Suspended Solids Analysis

Sample Stream	Aliquot Size (milliliters)
Return Sludge	2- 3
Mixed Liquor	10- 15
Final Clarifier Effluent	75-250

After filtration, the filters and aluminum pans were returned to the Milwaukee lab and dried overnight at 103°C. The following morning, the filters and pans were cooled in a dessicator for one hour and weighed. The one hour cooling time was considered quite critical and was adhered to as closely as possible.

All analyses were performed in duplicate and the arithmetic average of the duplicates was used in reporting the data.

It is difficult to establish measurement of variability in suspended solids analysis due to a lack of a standard for analysis. Volk(57), however, presented a technique for establishing confidence limits on a pair of measurements based on the expected mean difference between a pair of measurements. The 95% confidence range for the $\bar{X}$ of a pair of measurements is $\bar{X} \pm 1.23\bar{d}$ where $\bar{d}$ is the mean difference between a pair of measurements.

Based on the results of 100 duplicate analyses the mean difference $\bar{d}$ was found to be 0.94. Thus, the 95% confidence limit for the mean of two samples was found to be:

$$\bar{X} \pm 1.22 \times 0.94$$

$\bar{X} \pm 1.2$ mg/l for the effluent suspended solids

Sludge Blanket Depth

The depth of the sludge blanket in the final clarifiers was measured with a small hand-held bilge pump. A 14 foot, calibrated section of tygon tubing was attached to the suction side of the pump. Sludge blanket depth was then estimated by visual observation of the sludge pumped from various depths. It is estimated that the accuracy of this technique is about ± 0.5 feet.

Solids Sensor Tests

A more detailed analysis of the sludge deposition patterns and blanket depths was made using the Rex Chainbelt solids sensor.

The major coponents of the solids sensor are: (A) a solids sensing probe, (B) electrical converter, (c) recorder, (D) extendible boom, (E) drive motor, and (F) boom support.

The solids sensing probe consists of a waterproof electrical junction box, a coil assembly, a vibratory paddle, and a paddle guard cage.

The paddle is connected to the two coils located in the coil assembly by two rods. One of the coils is operated at a closely controlled frequency of 120.1 CPS. This vibrational energy is transmitted through one of the rods to the paddle. The amplitude through which the paddle will vibrate is dependent on the specific gravity of the media in which it is immersed. The other rod transmits this amplitude modulated vibration into the field of a permanent magnet located in the center of the other (pickup) coil. A voltage is generated, measured, and recorded which is related to the amplitude of this vibration. The sensor probe has been designed to operate over the specific gravity range of approximately 1.0000 to 1.2500.

The procedures for calibration and operation of the solids sensor were similar to those outlined by Albrecht et al(55).and will not be detailed in this report. This apparatus was used during the latter stages of the report to investigate some of the basic assumptions regarding solids deposition patterns and buildup.

Settling Rate and SVI

Settling velocities and the sludge volume index were determined using an unstirred 1 liter graduate. The position of the solids-liquid interface was observed at various time periods to establish the settling curve. A typical curve is shown in Figure 18. Prior to the onset of sedimentation, the solids in the graduate were uniformly dispersed by means of a 2 inch diameter circular disc attached to the end of a stirring rod.

The settling rate was established by determining the slope of the linear portion of settling curve (Figure 18). The sludge volume index was determined by dividing the settled volume of sludge at 30 minutes by the mixed liquor suspended solids concentration.

Additional studies were also performed with 4 inch and 4.75 inch diameter settling columns to determine the effect of cylinder diameter on the settling rate. The diameter of the cylindrical mixing disc was determined from the following relationship:

$$D_1^2/A_1 = D_2^2/A_2$$

where:

D_1 = diameter of cylindrical disc in 1 liter graduate

A_1 = surface area of 1 liter graduate

D_2 = diameter of cylinder

A_2 = surface area of cylinder

Dissolved Oxygen

Dissolved oxygen measurements were periodically made at various points in the aeration and reaeration tanks to assure that oxygen limiting conditions did not exist. These tests were made using a silver-lead galvanic cell with a KOH electrolyte contained by a polyethylene member. The probe was fabricated by Rex Chainbelt Inc. research personnel. The depolarizing effect of dissolved oxygen on the cell causes changes in probe output. The output was measured on a 0 to 25 microampere DC microammeter. The probe was calibrated in oxygen saturated secondary effluent.

Flow Adjustments and Sampling Procedures

A general procedure was developed for testing of the final clarifiers and was followed throughout this study unless specifically mentioned. The

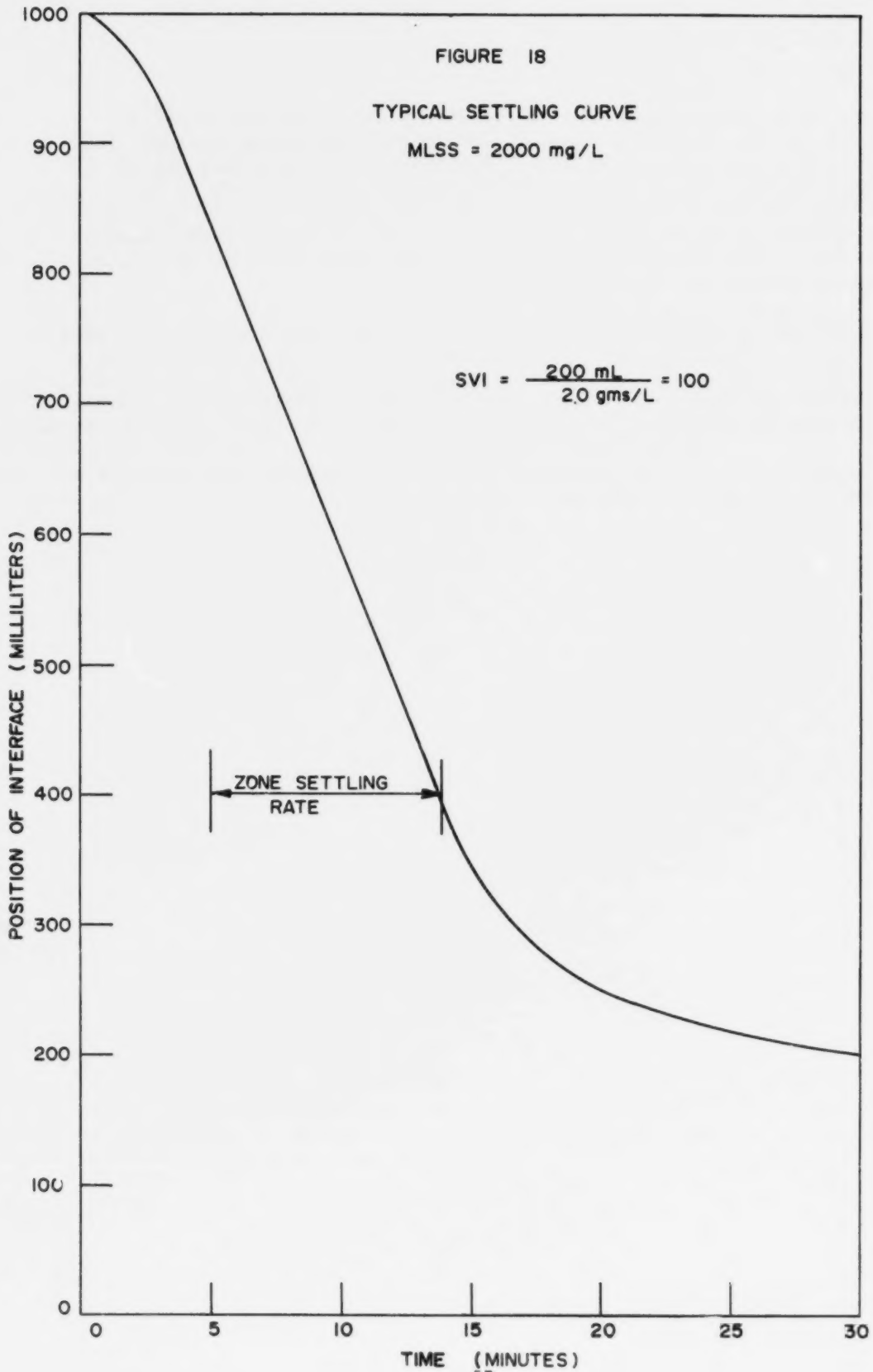


FIGURE 18
TYPICAL SETTLING CURVE
MLSS = 2000 mg/L
SVI = 200 mL / 2.0 gms/L = 100
ZONE SETTLING RATE
POSITION OF INTERFACE (MILLILITERS)
1000
900
800
700
600
500
400
300
200
100
0
0
5
10
15
20
25
30
TIME (MINUTES)

flows to be investigated were controlled by diverting the required portion of the total mixed liquor flow to the tank being studied. The return flow was then set at the proper rate using the techniques of Bloodgood(60). In the case where deep sludge blankets were to be studied, the return flow was turned off for a period of time to permit the blanket to build up. After all the conditions under investigation were set, a one hour stabilization time was observed before the collection of samples was instigated.

Samples were generally collected over a 3.5 hour period at one-half hour intervals. All flows from the basin were noted and recorded whenever a sample was collected. Effluent and mixed liquor samples were generally collected and analyzed for each sampling period. Return sludge samples were taken at the start, midpoint, and end of the 3.5 hour sampling period.

Settling rates, SVI, and other miscellaneous parameters were measured as often as physically practical.

RESULTS

The purpose of this project was to develop a mathematical model of a final clarifier for the activated sludge process. This model is to be incorporated into the FWPCA Preliminary Design and Simulation of Wastewater Renovation Systems Digital Computer Model. For this reason the model had to be formulated using only those parameters which would be available during preliminary design of simulation studies.

These parameters include the mixed liquor solids concentration, the raw waste flow and the BOD loading on the aeration tank. In order to use any additional parameters to evaluate final tank performance it is necessary to provide them to the model as input. An example of information which needs to be entered as input to the model would be the surface area and depth of the final settler. A minimum amount of input is desirable.

The selection of variables to be evaluated for possible inclusion in the model was based on the results of the literature review. A summary of these variables was given in Table 5.

Experiments were conducted at the three plants described in the Experimental Procedures Section in order to evaluate the performance of the final clarifiers. The data collected in these experiments are on file at the Robert A. Taft Sanitary Engineering Center, Cincinnati, Ohio. These experiments were based on a Box-Wilson experimental design procedure and were aimed at gathering data over a wide range of operating conditions.

Although seven half-hour interval sampling periods were monitored for each day of testing, only the daily average was included in the final data analysis. It was felt that this procedure would best reflect an average tank performance for a steady state analysis.

Analysis of Data

The data collected in this study were analyzed using statistical analysis packages available from the Service Bureau Corporation Call/360 Time Sharing Service and the General Electric Time Sharing Service.

These packages make available a large number of statistical techniques including scatter diagrams, elementary statistical analysis, multiple regression and step-wise multiple regression.

A general procedure of analysis included a scatter diagram to establish trends in the data, elementary statistical analysis to evaluate the mean, standard deviation and range of the data under investigation. This was followed by a step-wise multiple regression analysis.

Step-wise regression was used to select independent variables in the order in which they account for the variation in the dependent variable. Their accounting for variation is based on the reduction of sum of squares: the

independent variable which reduces the largest sum of squares in a given step is entered next into the regression(64). A detailed description of the statistical analysis techniques is available from the time sharing manuals(64)(65). An example of the output from the computer regression analysis is given in Appendix 1.

Development of the Models

The models developed for predicting the return sludge and effluent solids concentrations from a final settler will be presented in the following discussion. For purposes of clarity they will be presented separately although in reality the two functions cannot be separated.

Return Solids Concentration

A final clarifier in the activated sludge process is a solids separation and concentration device for which a mass balance must exist. For a clarifier which is designed and operated properly 95 to 100% of the solids entering the clarifier should be removed in the underflow. The remainder will escape in the effluent. Using the nomenclature expressed in Figure 19 this mass balance can be written as:

Lbs of Solids into Final Clarifier	=	Lbs of Return Sludge Solids	+	Lbs of Waste Sludge Solids	+	Lbs of Eff. Solids	±	Storage or Depletion

$$Q_4 \times 8.34 \times MLSS =$$

$$Q_6 \times 8.34 \times TSS_6 + Q_7 \times 8.34 \times TSS_7 + Q_5 \times 8.34 \times TSS_5 \pm \text{Storage or Depletion} \qquad (5)$$

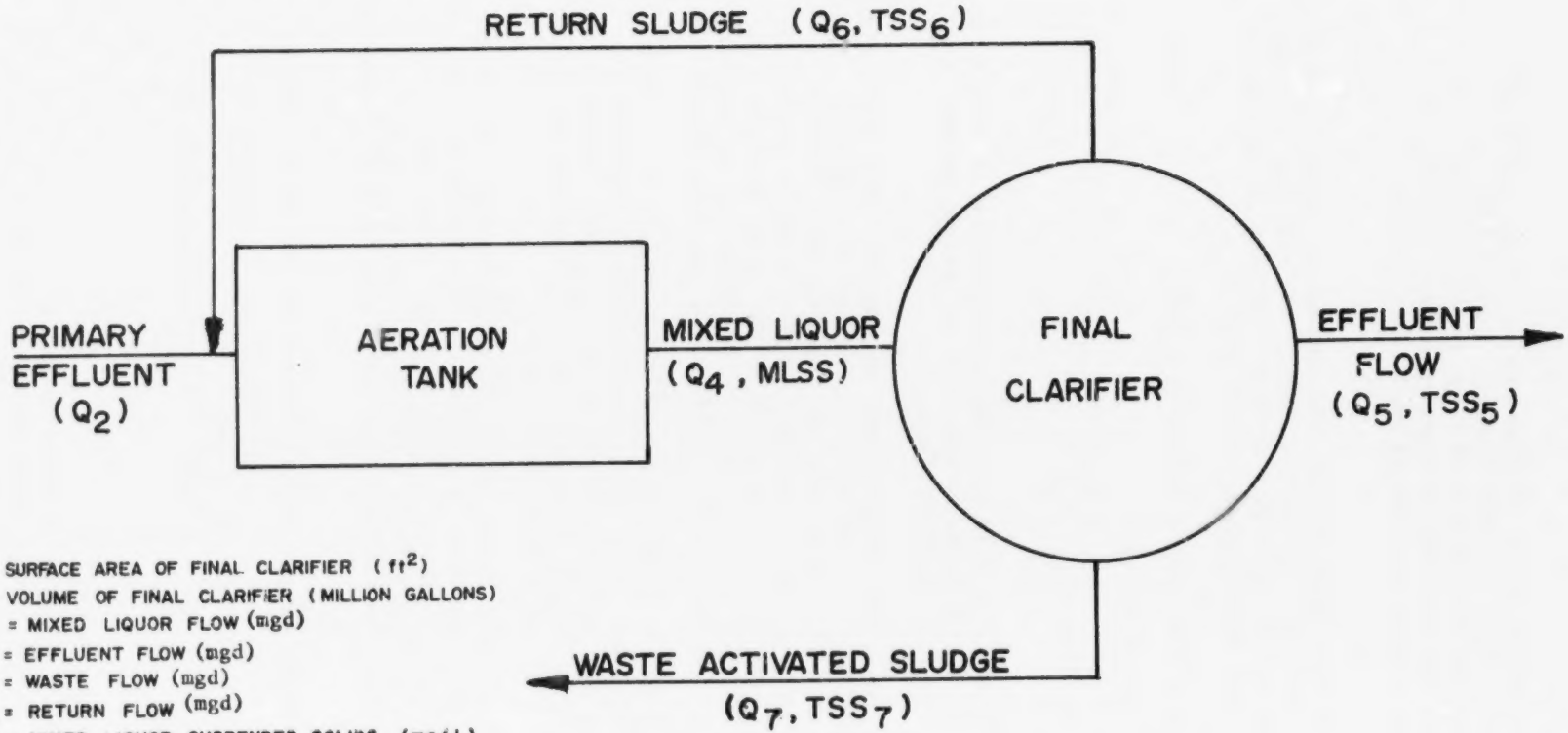
This equation is exact since it establishes the conditions necessary for a mass balance on a final settler. An equation can be written for the storage or depletion as follows:

$$S \text{ or } D = \pm A \times \Delta D_S \times \rho_S$$

where:

S = Storage (pounds)
D = Depletion (pounds)
A = Floor area of final settler (square feet)
ΔD_S = Incremental change in blanket depth (feet)
ρ_S = Density of sludge in increment of depth change (pounds/cubic feet)

Sludge storage in a final settler necessitates a rising sludge blanket. Sludge blanket depth must be controlled within certain limits to prevent the excessive discharge of solids in the effluent. For this reason, sludge storage in the final clarifier must be a short term operation. As such it is not seen as an important factor in the mass balance term except under severe nonsteady state conditions.



A = SURFACE AREA OF FINAL CLARIFIER (ft^2)
V = VOLUME OF FINAL CLARIFIER (MILLION GALLONS)
Q_4 = MIXED LIQUOR FLOW (mgd)
Q_5 = EFFLUENT FLOW (mgd)
Q_7 = WASTE FLOW (mgd)
Q_6 = RETURN FLOW (mgd)
MLSS = MIXED LIQUOR SUSPENDED SOLIDS (mg/L)
TSS_5 = EFFLUENT SUSPENDED SOLIDS (mg/L)
TSS_6 = RETURN SLUDGE SUSPENDED SOLIDS (mg/L)
TSS_7 = WASTE SLUDGE SUSPENDED SOLIDS (mg/L)

FIGURE 19

GENERAL FLOW SHEET FOR FINAL CLARIFIER

It is conceded that increased sludge blanket depths can be beneficial in providing a more concentrated return sludge. This benefit is, however, often overriden by deleterious effects on the performance of the activated sludge system (denitrification, phosphate leaching etc.). The maintenance of deep sludge blankets is most often reserved to gravity thickness.

At steady state and for most applications, the storage or depletion would be equal to zero. In addition, the solids lost in the effluent are small in proportion to the solids in the other streams. Setting the storage or depletion and the effluent solids equal to zero yields.

$$Q_4 \times MLSS = Q_6 \times TSS_6 + Q_7 \times TSS_7 \qquad (6)$$

$$\text{but } TSS_6 = TSS_7$$

$$\text{Therefore } TSS_6 = \frac{Q_4 \times MLSS}{Q_6 + Q_7} \qquad (7)$$

Inspection of this equation indicates that the return sludge concentration is directly proportional to the mixed liquor flow and solids content and inversely proportional to the underflow rate.

Although this equation is theoretically sound, it does not take into account the thickening requirements of the clarifier. Biological sludges do not concentrate to an infinite concentration. An underflow rate must be selected which will insure that a sufficient mass of solids is removed in the underflow to prevent the buildup of solids in the final settler. It thus becomes apparent that the return sludge concentration is a function of the mixed liquor input as well as the operation of the final clarifier.

In actual plant operation, the return rate can be controlled by a number of different methods. Bubbler tubes can be used to visually observe the sludge blanket depth and the rate can be set to maintain the blanket at a particular depth. Various methods of controlling sludge return rate by means of photoelectric cells have also been used.

Bloodgood(60) proposed a method of selecting the minimum return rate based on the sludge volume index. Although this procedure has recently been criticized(66) it is felt to be the only currently available procedure which can be used in a simulation model. For sludges which settle well such as at the Racine plant the procedure has been found to offer reasonable results.

Using Bloodgood's procedure, the percent return can be estimated from the following relationship:

$$P = \frac{SVI \times MLSS}{10^6} \qquad (8)$$

where:

$P = (Q_6 + Q_7)/Q_4$
SVI is the Sludge Volume Index, and
MLSS is the Mixed Liquor Suspended Solids in mg/l.

The use of this equation in a simulation or preliminary design model requires an estimate of the Sludge Volume Index. Quantitative relationships defining the SVI as a function of the activated sludge operational parameters are nonexistent. Historically, activated sludge mathematical models have either neglected the sludge subsidence and compaction characteristics or assumed a value with no concern for the consequences.

Data from the Hyperion, California plant were analyzed in an attempt to relate the SVI to various operational parameters of the activated sludge process. The data resulted from a special FWPCA - Hyperion study, the purpose of which was to evaluate various key parameters for the FWPCA activated sludge mathematical model. Since the data were obtained under different organic loading rates, detention times and cell residence times it presented the opportunity to attempt to relate SVI to the operational characteristics of the activated sludge system.

Data were available for nine different modes or conditions of operation from the Hyperion study. Those parameters which were thought to possibly influence SVI included, lbs BOD/day/lb MLSS, lbs BOD/day/#MLVSS, Mixed Liquor Aeration Time, Cell Residence Time, Temperature, and volatile solids concentration. The average value for each parameter was calculated for each mode of operation. These values were then subjected to a multiple regression analysis. The following empirical relationship is a result of this analysis:

$$SVI = 540 \times A^{4.397} \times B^{0.213} \qquad (9)$$

where:

A = Fraction of volatile suspended solids in the mixed liquor (% VSS/100)
B = BOD loading (lbs BOD/day/#MLVSS)

This equation had a multiple correlation coefficient of 0.914. Utilizing the mean values from each of the Hyperion modes of operation resulted in 9 values upon which this analysis is based. Each of the 9 values represents an average of from 5 to 12 individual observations. A brief statistical analysis of the raw data is shown in Table 9.

TABLE 9

Variable	Mean	Std. Deviation	Maximum	Minimum	Range
SVI	123.6	58.96	232	61.5	170.5
Percent VSS	73.0	5.78	81.97	64.7	17.3
lbs BOD/day/lb MLSS	0.513	0.302	1.00	0.207	0.79

The actual and calculated SVI's for the data used in the regression analysis are shown in Figure 20.

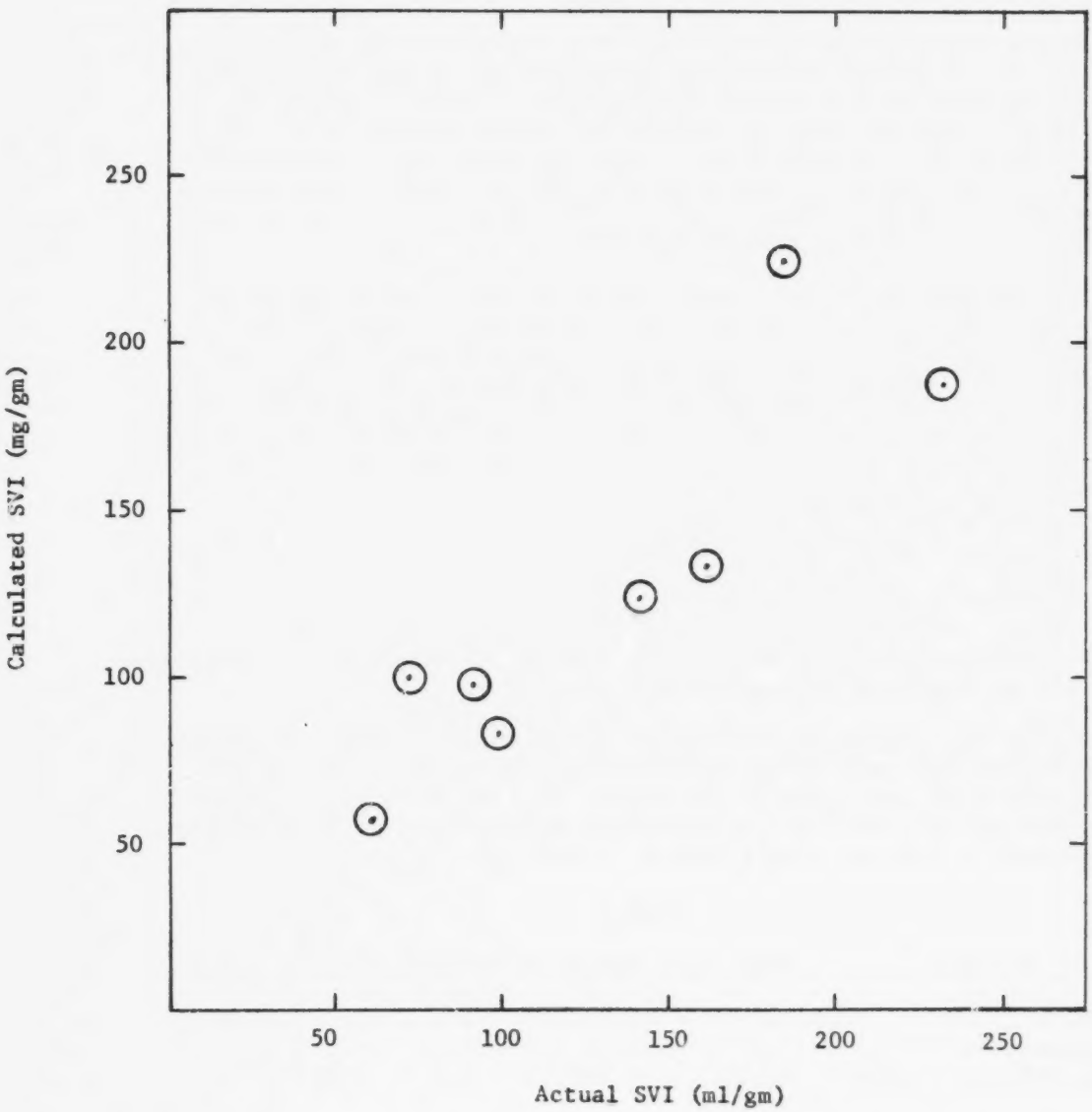


FIGURE 20 ACTUAL VS CALCULATED SVI's FOR HYPERION DATA

Equations 7, 8, and 9 can be combined to provide an estimate of the maximum possible return sludge concentration.

$$TSS_6(max)(mg/l) = \frac{10^6}{540 \times A^{4.397} \times B^{0.213}} \qquad (10)$$

Return sludge concentrations less than this maximum result from return sludge flow rates in excess of the theoretical rate given by equation 8.

The return sludge concentration at flows greater than theoretical is proportional to the ratio of the return sludge rates. In practice, the return sludge rate should be operated as close to the theoretical rate as possible in order to minimize sludge pumping costs. In addition, the effective raw waste detention time in the aerator can be increased when the minimum sludge return rate is used.

The use of equations 8, 9, and 10 can best be illustrated by the following example. It is desired to calculate the return sludge concentration from a final clarifier. The input mixed liquor concentration is 2000 mg/l, percent volatile solids is 75%, and the BOD loading is 0.4 lbs BOD/day/lb MLVSS.

The SVI is estimated from equation 9.

$$SVI = 540 \times 0.75^{4.397} \times 0.4^{0.213} = 125$$

The percent return is given by equation 8

$$P = \frac{125 \times 2000}{10^6} = 0.25$$

The mixed liquor flow can be determined from the following relationship:

$$Q_4 = Q_5 + Q_6 + Q_7$$

Since Q_7 is very small in proportion to the total flow (1-2%) it can be neglected and:

$$Q_5 = Q_2$$

$$\text{Also } \frac{Q_6 + Q_7}{Q_4} = P$$

$$\therefore Q_4 = Q_2 + (PQ_4)$$

and

$$Q_4 = \frac{Q_2}{(1-P)}$$

$$Q_4 = \frac{1.0}{(1 - .25)} = 1.33 \text{ MGD}$$

$$Q_6 = Q_4 - Q_2$$

$$Q_6 = 1.33 - 1.0 = 0.33$$

$$TSS_6 = \frac{10^6}{540 \times .75^{4.397} \times .4^{0.213}} = 7974$$

For return flow rates higher than the theoretical rates calculated in this example, the return sludge concentration will be proportionately decreased. It is felt that this approach is the best procedure for estimating the return sludge concentration with the information currently available to the design engineer. It is hoped that further research will yield a more reliable estimate for return sludge concentration.

It should be emphasized that the empirical SVI equation (equation 9) is by no means exact and is not meant to indicate a cause-effect relationship. It is used here only to provide an estimate of the SVI. It is hoped that future work will permit a more realistic approach to the selection of a return rate based on a more detailed investigation of the operational characteristics of the aeration basin such as BOD loading and sludge age and the various other sludge parameters which describe the compactibility of activated sludge.

Empirical Relationship

Additional work was conducted in an attempt to formulate an empirical relationship for the return sludge concentration.

The data collected at the Racine, Brookfield and Fort Atkinson plants were subjected to a multiple regression analysis in order to formulate an empirical expression for the return sludge concentration. The data from the Hyperion, California plant were also analyzed.

The variables which were considered important to the return sludge concentration included the MLSS, mass loading, SVI, sludge blanket depth and percent sludge withdrawal based on the mixed liquor flow ($Q_6 + Q_7/Q_4$). Multiple regression analysis was run on the data from each of the plants and for all the data combined. The results of this analysis together with the relative contribution of each variable are shown in Table 10.

No correlation between sludge blanket depth or SVI and the return sludge concentration was found. Attempts to correlate the various interacting factors also was fruitless. These variables were therefore not included in Table 10. Typical actual and calculated values for the return sludge concentration using the combination equation are shown in Table 11.

TABLE 10

Empirical Equations for Predicting Return Sludge Concentration

Plant Name	Equation	Equation Number	Multiple Corr. Coefficient	Proportion of Total Sum of Squares Reduced by Each Variable
Hyperion	$C_R = \frac{27.8\ (MLSS)^{.428} \times (ML)^{.432}}{P^{.896}}$	11	.977	MLSS - .926 ML - .001 P - .028
Racine	$C_R = \frac{6.18\ (MLSS)^{.656} \times (ML)^{.263}}{P^{.94}}$	12	.979	MLSS - .193 ML - .012 P - .752
Brookfield	$C_R = \frac{5.31\ (MLSS)^{.889}}{P^{1.22} \times ML^{.533}}$	13	.949	MLSS - .051 ML - .815 P - .032
Fort Atkinson	$C_R = \frac{.618\ (MLSS)^{1.16} \times ML^{.038}}{P^{.649}}$	14	.948	MLSS - .128 ML - .019 P - .754
Combination of 4 Plants	$C_R = \frac{2.57\ (MLSS)^{.84} \times (ML)^{.12}}{P^{.88}}$	15	.971	MLSS - .149 ML - .722 P - .073

Where: MLSS = Mixed liquor suspended solids mg/l
ML = Mass loading lbs. MLSS/day/ft^2
P = Percent return

TABLE 11

Actual and Calculated Values for Return Sludge Concentration

Return Conc. Actual mg/l	Return Conc. Cal. mg/l	Return Conc. Actual mg/l	Return Conc. Cal. mg/l
6000	5432	8792	9886
5800	4533	5172	5317
5900	5041	12023	10842
5600	5693	16221	17916
6500	5296	18292	19128
7300	7589	6920	6299
7700	7121	9416	8071
7200	7347	9966	8388
8100	6843	11876	12468
8100	7562	5274	5779
8100	7980	20175	14376
8000	7773	6280	5878
8100	7776	5280	4786
7400	6984	5278	4801
7200	6843	6595	6346
6800	6909	5507	5428
7500	7290	9740	12042
7500	7050	10400	8397
7500	7062	26962	28486
5700	5607	8113	7571
5700	5465	19565	18353
6100	5808	1763	13524
6500	5796	.63	21003
5900	5487	8250	8335
5800	5379	16263	14781
5900	5716	8075	9392
6100	5660	21500	19187
4700	4583	5575	6462
5300	5170	19450	18871
3460	3050	20625	19054
3820	3559	5768	6493
3100	2852	17250	15211
4060	3180	4642	5382
3760	3180	32600	25515
3540	3097	7040	8000
3120	2952	7180	7650
2500	2509	7350	7972
2700	2753	6450	6684
2900	2569	6380	7129
2300	2560	5430	5990
3800	3076	5335	5555
2800	2523	5670	6166
2200	3001	6430	7528
3200	3001	6060	6912
2800	2864	6190	6734
3300	2864	6180	7529
9346	8879	5320	6426
7367	6856	5540	6378
7633	6959	6430	7526
3045	3844	6060	6910
10235	9124	6190	6734
5848	6167	6180	7061
14265	10646	5130	5770
6750	6339	5320	6447

It can be seen from Table 10 that different variables make the most significant contribution at the different plants. For example, MLSS reduces the total sum of squares by 92.6% for the Hyperion data but only 19.3 for Racine and 5.1% for Brookfield. Similarly, the mass loading reduces the total sum of squares by 0.1% for Hyperion but 1.2% for Racine and 81.5% for Brookfield. The reason for this is thought to be the design and operation of the final clarifiers. For example, at the Racine and Fort Atkinson plants, the sludge settled very rapidly so it might be expected that the percent return would be controlling variable. However, at the Brookfield plant the sludge did not settle well and the area required for thickening may have been the limiting factor.

The regression analysis on the combined data from the four plants is a general equation derived from four different sludges, rectangular and circular basins and a hydraulic and mechanical sludge collector. It is thought to give the best estimate of return sludge concentration of all the possible empirical formulations investigated. This empirical formulation can, however, be mathematically rearranged to yield essentially a mass balance type of equation similar to equation 7. It is, therefore, probably more meaningful to use the procedure outlined earlier in this section to calculate return sludge concentration.

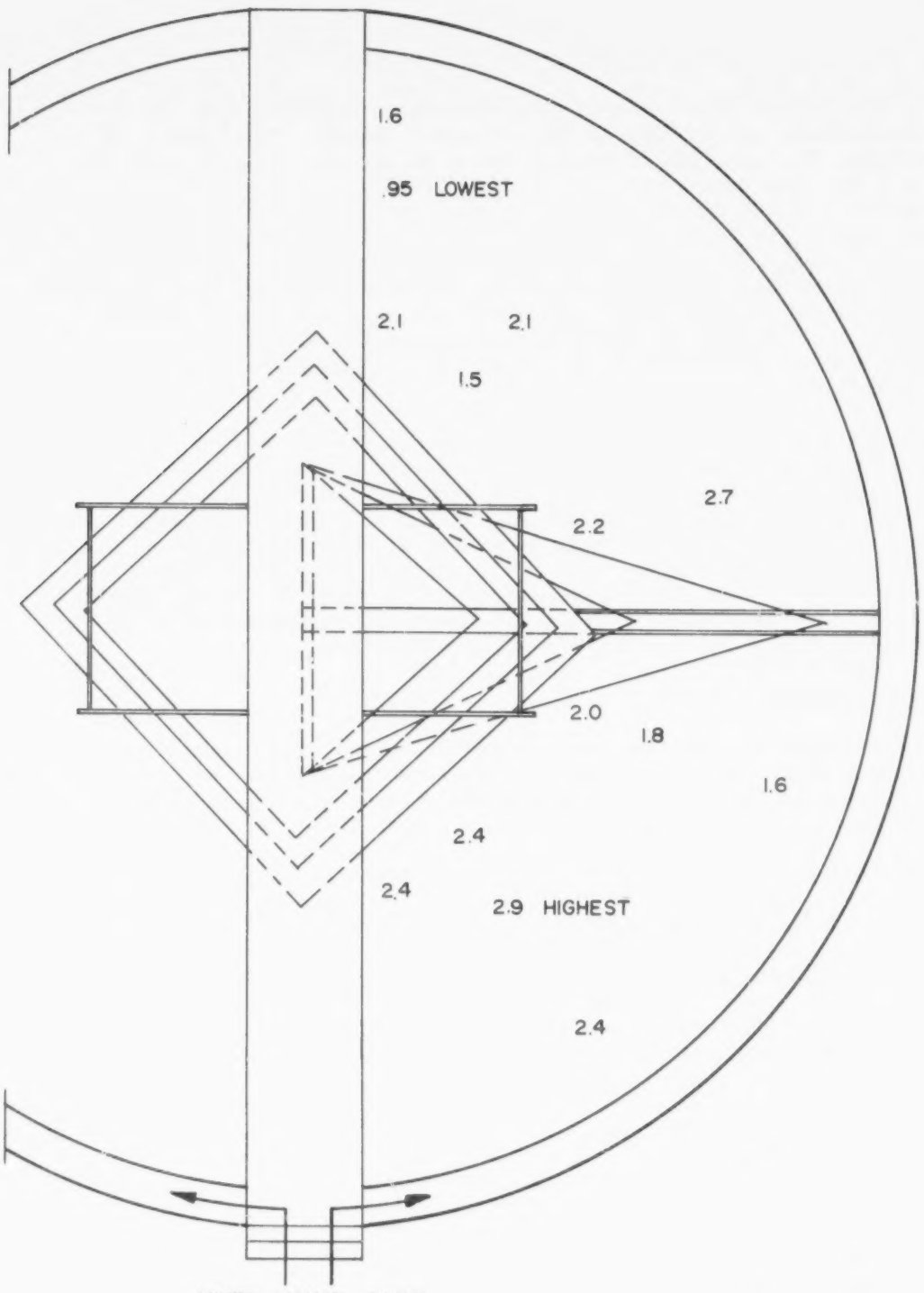
Solids Sensor

The Rex solids sensor was used in an attempt to investigate the deposition pattern of sludges in a final clarifier and the solids profiles at various positions in the tank. Of particular interest was the maximum sludge concentration which could be achieved at the bottom of the clarifier.

An analysis of the solids inventory in a final clarifier is a very complicated task because of the large area to be analyzed. In order to facilitate this analysis only one-half of the tank was studied.

Figures 21 and 22 show the solids inventory expressed in pounds of dry solids/day/foot^2 at various positions in the tank. All values were determined at a 2 to 3.5 minute interval after the passage of the sludge collector. As a general pattern it can be seen that the highest solids inventory was located closest to the tank inlet. In the absence of dye studies it is impossible to determine whether this is due to an uneven distribution of flows or the result of the heavier solids settling out in the inlet channel. It should be pointed out that the activated sludge at the time of this study was approximately 50% volatile with an SVI of 32. This sludge is not typical of most activated sludges.

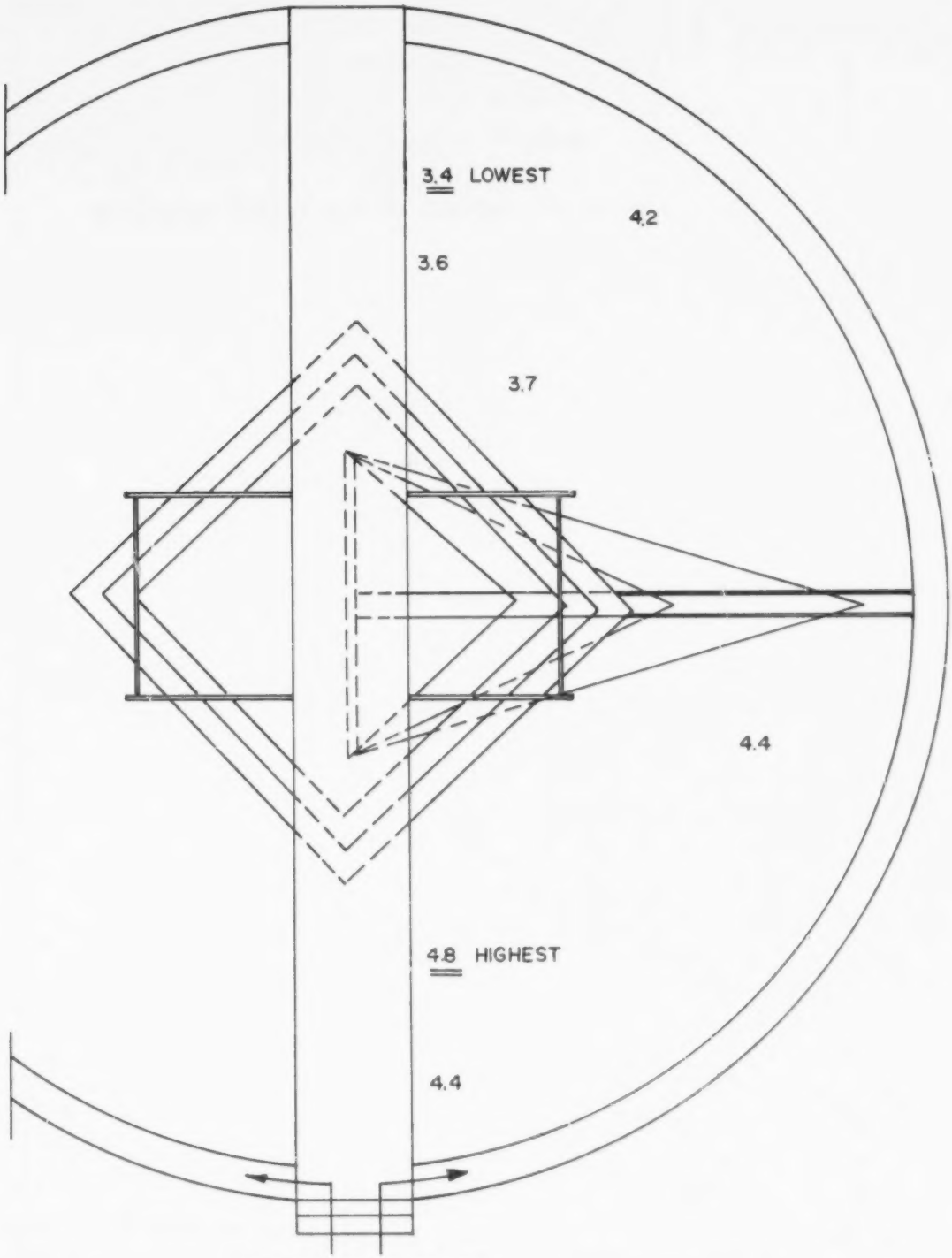
In Figure 23 the solids concentration at various times after the passage of the sludge collector is plotted. The sludge concentration increased from a minimum immediately after the sludge collector had passed to a maximum just before the passage of the next collector arm.



14 APRIL 1969

FIGURE 21

SOLIDS DEPOSITIONS PATTERNS IN lbs DRY SOLIDS PER SQ. FOOT, MEASURED APPROX. 2 MIN. AFTER TOWBRO PASSAGE



10 APRIL 1969

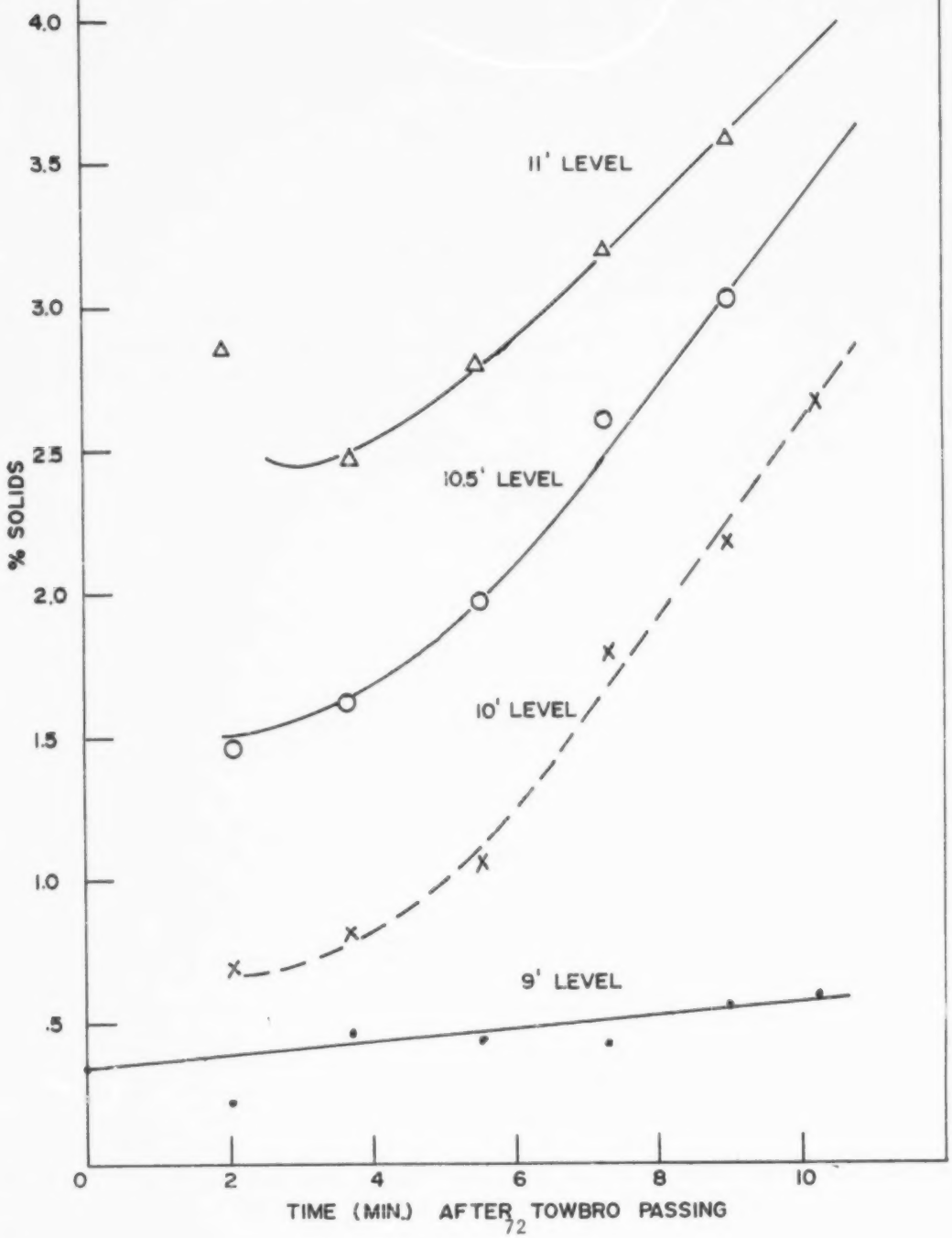
FIGURE 22

SOLIDS DEPOSITION PATTERNS IN lbs, DRY SOLIDS PER SQ. FOOT
MEASURED APPROX. 3.5 MIN. AFTER TOWBRO PASSAGE

FIGURE 23

RACINE

PERCENT SOLIDS CONCENTRATION
VERSUS
TIME AFTER PASSAGE OF THE SLUDGE COLLECTOR



The solids concentrations at the various tank depths indicate the solids profile and clearly show the thickening process in the clarifier. A comparison of the maximum sludge concentration at the 11 foot depth, the approximate depth of sludge withdrawal, and the theoretical concentration derived from the laboratory settling test can be made. The theoretical concentration derived from the laboratory settling test was 28.7 grams/liter. The actual concentration at this withdrawal point in the tank was about 35.5 grams/liter. The actual return concentration was about 30.0 grams/liter. Whether this discrepancy is true for the whole tank or only at the single point tested cannot be proved because point analysis could not be conducted in the inner 20 feet of the tank radius due to the complicated underwater structural members. It is reasonable to assume that with this fast settling sludge, a differential deposition pattern exists. This would mean that the inner section of the hydraulic sludge collector would be drawing a slightly more dilute sludge than the outer sections.

Since the information available from the use of the solids sensor was not of great use in the formulation of the model, only a limited amount of tests were made with the sensor. It is felt that the greatest potential use of this apparatus would be in the analysis of gravity thickeners and final clarifiers with deep sludge blankets.

Effluent Suspended Solids

The primary function of a final clarifier is to provide an effluent which is low in suspended solids. Since the effluent stream, in most cases, represents the final step in secondary waste treatment, the performance of the final settler is of primary importance in the overall treatment scheme.

An attempt has been made to develop a mathematical relationship which will predict the effluent suspended solids given the mixed liquor flows, solids concentration and the operational parameters of the aeration tank.

Based on the results of the literature survey and an understanding of the sedimentation process, it would appear that the effluent solids from a final clarifier are a function of the hydraulics of the tank and of the sludge quality. This relationship is expressed in the following equation:

Effluent Suspended Solids	= f	overflow rate, detention time, sludge return rate, weir loading, mass loading
	+	subsidence characteristics, flocculation characteristics, active bacterial mass, shear imparted in aeration, inert solids in the activated sludge, particle size distribution

Analysis of this equation indicates that for a fixed sludge quality, the effluent suspended solids are a function only of the hydraulics of the basin. Basin hydraulics factors are commonly measured and are defined below.

The overflow rate of the final clarifier is defined as:

$$ORA = \frac{Q_5}{A} \tag{16}$$

Where ORA is the actual tank overflow rate (gallons/day/ft^2), Q_5 is the effluent flow gallons/day and A is the surface area of the final settler.

The detention time is based on the mixed liquor flow and is defined by:

$$T = \frac{V}{Q_4} \times 24 \tag{17}$$

Where T is the tank detention time in hours, V = tank volume in million gallons and Q_4 is the mixed liquor flow in MGD.

The mass loading is defined as the pounds of mixed liquor solids per square foot of tank area per day.

$$ML = \frac{Q_4 \times 8.34 \times MLSS}{A} \tag{18}$$

Where:

ML = mass loading (pounds mixed liquor solids/day/foot^2)

Q_{ml} = mixed liquor flow (MGD)

C_{ml} = mixed liquor suspended solids (mg/l)

A = tank surface area (sq ft)

The weir loading is given by equation 20.

$$WL = \frac{Q_5}{L} \tag{19}$$

Where: Q_5 is the effluent flow (gallons/day) and L is the length of weir (feet).

These parameters can be used to characterize the hydraulic flows within a basin.

Multiple regression analysis was performed on the data collected at each of the plants in an attempt to find a relationship to predict the effluent suspended solids from a final clarifier. Scatter diagrams were used to indicate the form of the equations. In most cases no definite trends were obvious.

Throughout the course of this study it has been found that when data collected over a short period of time are analyzed a relatively good fit to the data can be made with a multiple regression analysis. An example of this was seen at the Racine plant. When the data from the period of April 10, 1969 to May 21, 1969 were subjected to a regression analysis, an equation of the form:

$$\text{Eff. SS} = 18.2 + .0136 \times \text{ORA} - .0033 \times \text{MLSS} \qquad (20)$$

This equation had a multiple correlation coefficient of 0.91 based on 47 observations.

However, when data collected over a longer period of time were analyzed the fit of the equation decreased significantly. It is felt that the reason for this decrease in fit is a basic change in the composition of the sludge which is unaccounted for by a gross parameter such as mixed liquor solids concentration.

Various attempts to provide a measurement of sludge characteristics using parameters such as aeration time, sludge age and BOD loading have not proven exceptionally fruitful.

A multiple regression analysis of the data from the Racine and Hyperion plants has shown some promise for the predication of effluent suspended solids. The equation is shown below:

$$\text{Eff. SS} = \frac{382 \times \text{OR}^{.12} \times (\#\text{BOD/day}/\#\text{MLSS})^{.27}}{\text{MLSS}^{.35} \times \text{DT}^{1.03}} \qquad (21)$$

This equation has a multiple correlation coefficient of 0.63. The results of a statistical analysis on the input data are shown in Table 12. A plot of the actual and calculated values is shown in Figure 24.

TABLE 12

Statistics for Input Variables for Regression Analysis

Variable	Mean	Std. Deviation	Max.	Min.	Range
Eff. SS	19.84	11.95	66	3	63
O.R.	734	364	1990	115	1875
#BOD/day/#MLSS	.423	.313	1.49	.044	1.45
MLSS	2128	1026	4437	500	3937
D.T.	2.84	1.075	4.37	.91	3.46

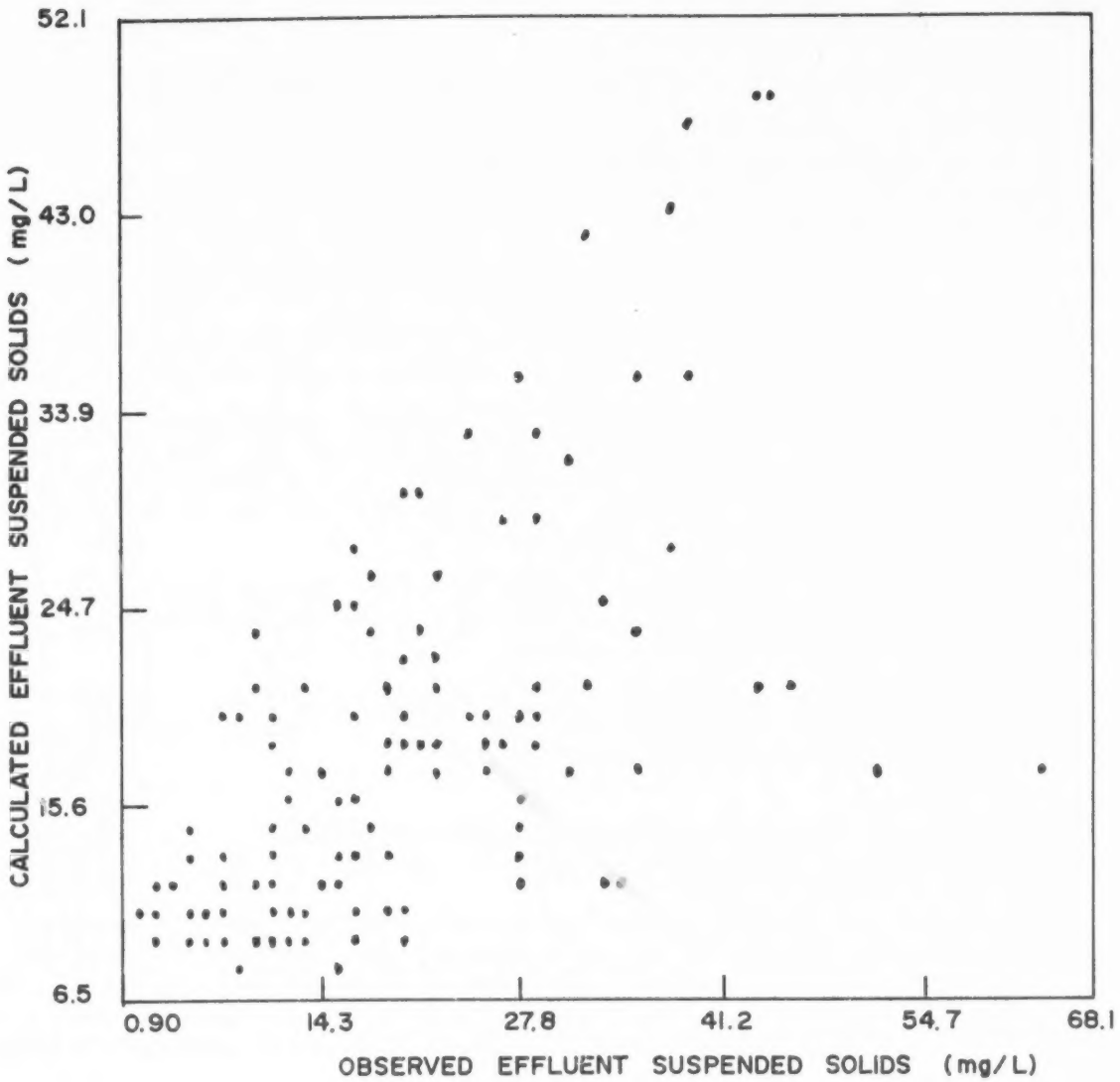


FIGURE 24

OBSERVED AND CALCULATED EFFLUENT SUSPENDED SOLIDS

This equation is felt to offer the best results over a broad range of input variables of any of the equations tested in this study. The equation is proposed, therefore, as a preliminary model to predict effluent suspended solids under steady state conditions.

Additional work is required to improve the ability of this equation to predict effluent suspended solids. The following discussion will include suggestions for future work.

The results of attempts to develop an equation to predict the effluent suspended solids have shown the need for further research. It is felt that a more complex relationship describing the sludge quality is necessary to adequately characterize the sludge. Sludge quality will be discussed in a later section.

Two factors which are felt to influence the effluent suspended solids over which no control was exerted were the scum and colloidal materials.

The amount of scum materials on a final clarifier is a function of the efficiency of the primary sedimentation facilities and the operational characteristics of the treatment plant. Scum materials do not flow from a final clarifier as a function of flow rate. Moreover, their separation paths deviate from that of the activated sludge solids, and their magnitude may be of the same order as the effluent suspended solids. They tend to accumulate on the leeward side of the tank and are generally carried from the tank immediately following a shift in the wind. Thus, depending on the time that the sample is collected, the suspended solids due to scum materials may or may not influence the results. The problem of floating materials on final clarifiers has become more serious in recent years. For example, with the advent of biodegradable LAS, an increase in floatable materials is being encountered in the final clarifier. It is felt that the influence of scum materials is responsible for part of the difficulties encountered in obtaining a good correlation.

Colloidal materials are present in raw sewage and are generated during the biological treatment process. Colloids are by definition particulate matter smaller than 0.2 - 0.5 microns. Since this is in the range of the pore size openings, it is not unreasonable to expect that some colloidal materials will be captured on the filter during the suspended solids analysis. This additional weight contributed by colloidal materials which are not by definition suspended solids can account for some of the unexplained variance in the regression model. Colloidal materials should be defined by the sludge quality parameter.

This equation neglects the hydraulic effectiveness of the various basins studied. Although hydraulic effectiveness is felt to play an important role in the performance of the final clarifier, it is impossible to investigate except under side by side testing with different tanks and the same sludge. Hydraulic effectiveness needs further investigation and should eventually be incorporated into the model.

DISCUSSION OF RESULTS

The formulation of a series of equations to predict the performance of a final clarifier has been presented. As the result of the extensive effort to develop these equations, a certain amount of qualitative information and observations which cannot be expressed in mathematical terms have evolved. This information will be presented in this section together with recommendations for the improvement of the model.

Mathematical models of the activated sludge process are abundant and a tremendous amount of research effort has been expended in order to develop the biochemical theories and to measure the reaction rates for the various process modifications and waste characteristics. The majority of this work has ignored the effect of the sedimentation function of the process and has concentrated only on the biochemical response. As a result there exists almost no information on solids separation characteristics of various activated sludges.

It should always be remembered that the activated sludge process consists of two interrelated functions, aeration and sedimentation. There is no way to separate these functions as what happens in sedimentation and vice-versa. For example, an activated sludge model can predict the effluent soluble BOD from an aerator under given loading conditions but the return sludge rate might have to be 100% of the raw flow to attain the desired biological solids concentration. This high return rate will then decrease the detention time in the aerator to one-half the raw waste detention time and undoubtedly will influence the biochemical results.

Thus, mathematical models of the activated sludge process should be concerned with the subsidence characteristics of the sludge as well as the biochemical reaction rates. In order for an activated sludge model to be truly useful it should give some estimates of the subsidence properties of the sludge based on the operational parameters of the activated sludge system.

The harmonious operation of the clarification and thickening functions of a final settler requires a knowledge of the sludge characteristics. It has been shown that with relatively constant sludge characteristics the effluent solids from a final clarifier can be described by the overflow rate and mixed liquor solids concentration (equation 20).

As a result of observations made during this study, a new parameter, sludge quality, will be defined. Sludge quality is an expression defining those properties of an activated sludge which determine its settleability and effluent clarity.

Activated sludge is made up of active microbial solids, the biodegradable volatile solids in the primary effluent, the nonbiodegradable volatile suspended solids in the primary effluent, the nonbiodegradable suspended solids resulting from endogenous respiration, and the inert nonvolatile solids in the primary effluent resulting from chemical precipitation in the aeration tank. Although classification of these solids is a formidable job, some of the latest activated sludge models(67) have made an attempt to perform this function.

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An example for this type of approach was seen during the Racine testing program when the activated sludge system was switched from contact stabilization to the Kraus process. The introduction of the anaerobic digester supernatant into the activated sludge system resulted in a sludge which settled very rapidly. However, a fration of very fine digested solids which did not settle well began to appear in the effluent from the final settler. A solids classification system would account for these solids and would hopefully improve the prediction of effluent suspended solids.

The data presented by Keefer(24), Figure 8, appears to confirm this observation. Sludges which settle poorly and consequently have a high sludge volume index generally have a large area to volume ratio. If the tank overflow rate is low enough to prevent the gross carryover of solids, a very high effluent quality is generally observed. Gross measurements such as MLSS do not account for this quality of the sludge and make the prediction of effluent suspended solids difficult, if not impossible.

It is recognized that biological predomination plays an important role in the settleability of a particular sludge. However, this can generally be controlled by assuring optimum environmental conditions in the aeration tank.

Observation of the laboratory settling rate of a number of different sludges casts some light on the necessity for a sludge quality parameter. The supernatant clarity after the sludge interface has passed a particular position in the graduate is markedly different for various sludges. Although the laboratory settling rate describes the removal of the bulk of the sludge solids, it is those solids which remain in the supernatant after the passage of the interface which become effluent solids in a final clarifier.

It appears that these supernatant solids are fine particles which have escaped both flocculation and/or entrapment with the bulk of the sludge.

Since it is reasonable to expect that the different classes of materials comprising an activated sludge will have different flocculation and settling properties, it is felt that the classification of activated sludges according to their various components will allow a more accurate prediction of the effluent solids from a final clarifier.

One additional parameter which might be useful in the definition of the sludge quality is the zeta potential. During the early phases of this project some zeta potential measurements were made to determine its influence on effluent suspended solids from a final clarifier. These efforts were abandoned, however, since the amount of work required to develop this variable into a useful parameter to use in the math model for a final clarifier far exceeded the work scope of the project. In addition, it would be necessary to correlate zeta potential to the operational parameters of the aeration tank in order for this parameter to have any use in simulation or design models.

Samples of mixed liquor supernatant from the Milwaukee Sewage Treatment Plant were analyzed for zeta potential. Samples were taken at the start and end of the aeration tank. These data are summarized in Table 13.

TABLE 13

Zeta Potential Before and After Aeration Milwaukee Sewage

Date	Median Zeta Potential		Specific Conductance	
	Before Aer.	After Aer.	Before Aer.	After Aer.
11-11-68	-14.9	-14.8	1150	835
11-14-68	-17.0	-13.6	1210	950
11-15-68	-10.7	-12.1	830	720
11-18-68	-14.4	-12.6	1150	835
11-19-68	-12.7	-12.3	1100	960
11-20-68	-11.5	-10.3	1200	935

In all cases except one, there was a reduction in zeta potential as a result of aeration. It should be pointed out that this analysis was made on grab samples with no allowance for the lag time in the aeration tank.

Based on the few analyses which were made, it is felt that zeta potential does play a role in the clarification of activated sludge. However, a study of a very basic nature needs to be performed to evaluate zeta potential measurements. In order to be useful in a mathematical model, zeta potential would need to be correlated to some parameter such as BOD loading, sludge age, etc. Zeta potential is a parameter which should be investigated in a laboratory study where close controls can be exerted. If correlation between zeta potential and the flocculation of activated sludges, particularly the fine particulate matter left in suspension can be demonstrated, then this parameter would be extremely useful in predicting the effluent solids from a final clarifier.

The definition of the sludge quality as a function of the operational parameters of the activated sludge system is seen as a very important requirement to the improvement of the effluent solids prediction equation.

Preliminary Design

One of the original objectives of this project was to formulate a basis for the preliminary design of final clarifiers. As work progressed on the project it became increasingly obvious that this objective was impossible to meet utilizing the available information.

Clarifiers are normally designed with some knowledge of the subsidence characteristics of the sludge. The techniques for determining the limiting area were outlined in the experimental development section. The largest area required for either thickening or clarification is selected as the design area. This procedure is felt to be the best design procedure currently available.

However, this procedure is absolutely useless for the purposes of preliminary design since no knowledge of the subsidence characteristics of the sludge are available. The only remaining possibility is the use of design guidelines outlined in the Ten States Standards(30) or the Water Pollution Control Federation's, "Sewage Treatment Design Manual"(31).

In order to utilize a rational design approach, it is necessary to develop a mathematical relationship which relates the subsidence characteristics of activated sludges, such as settling rate, to the solids concentration, sludge quality and other operational characteristics of the aerator.

A suggested approach to this relationship would be to evaluate the subsidence characteristics of activated sludges under various conditions of BOD loading, sludge quality, and activated sludge modification. The only possibility for a realistic approach to the preliminary design of final clarifiers lies in the development of this relationship.

Settling Rate of the Solids

Settling rate has been shown to be an important variable affecting the performance and design of final clarifiers. For example, if the effluent flow upward (overflow rate) is greater than the solids settling rate, effluent quality will be adversely affected. Discussion regarding the effect of initial depth and cylinder diameter on the determination of the laboratory settling rate was presented in the literature search. Because this variable is thought to play such an important role in the performance of final clarifiers, the effect of cylinder diameter was investigated.

Side-by-side tests of a one liter graduate and a 4.75" diameter, 1 foot deep cylinder were run on sludges from three different plants.

Representative settling curves were selected and the settling rates are presented in Table 14. For dense sludges such as at Racine, and for dilute sludges from the Milwaukee plant, no consistent trend was noted when comparing the two settling rates. For the Brookfield sludge which did not settle well, 3 of the 4 settling tests indicated a higher rate in the larger diameter cylinder than in the liter graduate.

Based on these studies it can be concluded that cylinder diameter does have an effect on the settling rate of the solids. Future work is required to develop a standard laboratory settling test which represents as closely as possible the settling rate of the solids in a full size final clarifier. Only with a realistic estimate of the settling rate can a final clarifier be properly designed.

TABLE 14

Effect of Test Cylinder Diameter on Settling Rate

Racine Sludge			Milwaukee Sludge			Brookfield Sludge		
Settling Rate (ft/hour)		MLSS (mg/l)	Settling Rate (ft/hour)		MLSS (mg/l)	Settling Rate (ft/hour)		MLSS (mg/l)
1 Liter	4.75" Dia.		1 Liter	4.75" Dia.		1 Liter	4.75" Dia.	
36.9	-	2130	1.34	-	2920	4.1	7.70	1600
-	35.3	1920	-	6.9	2870	-	-	-
31.9	26.1	3200	18.95	-	1490	8.06	9.38	1575
18.5	-	4620	-	17.15	1450	7.3	6.60	1545
-	17.1	4570	27.5	-	890	-	-	-
21.95	16.45	3400	-	28.6	885	4.42	9.52	1585
-	-	-	26.0	-	535	-	-	-
-	-	-	-	35.3	495	-	-	-

SUMMARY

The purpose of this project has been to develop a series of equations to predict the performance and preliminary design requirements of a final clarifier in the activated sludge process. Equations to predict the overflow and underflow solids concentrations have been developed. The limitations of these equations have been discussed, together with suggestions for future research efforts to improve the equations.

The currently accepted design procedures for final clarifiers requires a knowledge of the settling rate of the sludge. Since no method is currently available for estimating the settling rate from the solids characteristics and the operational parameters of the activated sludge system, it is felt that the only alternative is to use the Ten States Standards(30) for preliminary design purposes in the model.

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APPENDIX 1

Example of Output from Computer Regression Analysis

```
Load ***Statpack
Ready

Infile Refile
Ready

Run

Statpack    11:07    08/09/68    Friday    SJ

Note:  No outfile - data cannot be saved

Are you a statpack expert
?* No
Do you wish to list names of available analyses
?* No

What analysis do you wish to perform
?* Stepwise regression

Do you wish to enter data from your input file now
?* Yes

 30 rows,  6 columns (infile)

Last data item read from input file

Do you wish to print the data just read in
?* No, this is the same data used in the regression section

Do you wish to change some values
?* No

Specify the dependent variable
?* 6

Do you wish to delete any variables
?* No


Step  1

Variable selected..... 2

Sum of squares reduced in this step.....      10.300
Proportion of variance of Y reduced.....       0.178
F for this variable (D.F.=1,  28).......       6.063

Cumulative sum of squares reduced.......      10.300
Cumulative proportion reduced...........       0.178  of      57.867

Multiple correlation coefficient........0.422    (Adjusted R =  0.422)
F for analysis of Vari. (D.F.= 1,  28)  6.063
Standard error of estimate..............1.303    (Adjusted SE=  1.303)
```

Variable	Reg.Coef.	Std.Error-Coef.	Computed T	Beta Coef.
2	0.00521	0.00212	2.462	0.42189

Step 2

Variable selected..... 5

Sum of squares reduced in this step..... 13.324
Proportion of variance of Y reduced..... 0.230
F for this variable (D.F.=1, 27)..... 10.506

Do you wish to enter this variable in the regression
?* SOS, are there any suggestions

In a given step, the variable that reduces the largest amount of sum of squares is selected. If the reduction indicated by the above 3 lines is significant, enter this variable in the regression. Otherwise, selection of variables will be terminated.

Do you wish to enter this variable in the regression
?* Yes

Cumulative sum of squares reduced..... 23.624
Cumulative proportion reduced..... 0.408 of 57.867

Multiple correlation coefficient..... 0.639 (Adjusted R = 0.622)
F for analysis of vari.(D.F.= 2, 27)... 9.314
Standard error of estimate..... 1.126 (Adjusted SE = 1.145)

Variable	Reg.Coef.	Std.Error-Coef.	Computed T	Beta Coef.
2	0.00632	0.00186	3.397	0.51162
5	0.04316	0.01332	3.241	0.48817

Step 3

Variable selected..... 3

Sum of squares reduced in this step..... 7.572
Proportion of variance of Y reduced..... 0.131
F for this variable (D.F.=1, 26) 7.382

Do you wish to enter this variable in the regression
?* Yes

Cumulative sum of squares reduced..... 31.196
Cumulative proportion reduced..... 0.539 of 57.867

Multiple correlation coefficient..... 0.734 (adjusted R = 0.711)
F for analysis of vari.(D.F.= 3, 26).... 10.137
Standard error of estimate..... 1.013 (adjusted SE = 1.050)

Variable	Reg.Coef.	Std.Error-Coef.	Computed T	Beta Coef.
2	0.00744	0.00172	4.318	0.60233
5	0.05363	0.01258	4.263	0.60648
3	0.01497	0.00551	2.717	0.38618
Intercept	-5.53531			

Step 4

Variable selected..... 1

Sum of squares reduced in this step.... 0.127
Proportion of variance of Y reduced.... 0.002
F for this variable (D.F.= 1, 25)..... 0.120

Do you wish to enter this variable in the regression
?* No

Do you wish to print the table of residuals
?* Yes

Case No	Y Observed	Y Estimated	Residual	Std.Resid.
1	1.000	0.599	0.401	0.396
2	2.000	1.884	0.116	0.115
3	2.000	2.266	-0.266	-0.263
4	0.0	0.907	-0.907	-0.896
5	2.000	1.998	0.002	0.002
6	2.000	1.584	0.416	0.411
7	3.000	3.499	-0.499	-0.492
8	2.000	2.233	-0.233	-0.231
9	3.000	3.859	-0.859	-0.848
10	0.0	0.989	-0.989	-0.977
11	4.000	2.513	1.487	1.469
12	1.000	1.959	-0.959	-0.947
13	1.000	2.050	-1.050	-1.037
14	1.000	1.107	-0.107	-0.106
15	3.000	2.920	0.080	0.079
16	2.000	1.765	0.235	0.232
17	3.000	2.541	0.459	0.454
18	4.000	3.366	0.634	0.626
19	4.000	3.680	0.320	0.316
20	3.000	2.654	0.346	0.341
21	4.000	3.700	0.300	0.296
22	4.000	1.846	2.154	2.126
23	1.000	2.069	-1.069	-1.055
24	0.0	1.956	-1.956	-1.932
25	4.000	1.340	2.660	2.626
26	1.000	1.798	-0.798	-0.788
27	3.000	2.245	0.755	0.745
28	4.000	4.413	-0.413	-0.407
29	4.000	3.926	0.074	0.073
30	0.0	0.333	-0.333	-0.329

Test of extreme residuals
Ratio of ranges for the smallest residual..... 0.263
Ratio of ranges for the largest residual..... 0.316
Critical value of the ratio at alpha = .10 ..... 0.332

Do you wish to plot Y observed and Y estimated
?* No

Do you wish to compute more regression
?* No

```
What analysis do you wish to perform
?* Finish
End of run

Time    0 min.    1 secs.
```

SELECTED WATER RESOURCES ABSTRACTS
INPUT TRANSACTION FORM

1. Report No.

2.

3. Accession No.
W

4. Title
A MATHEMATICAL MODEL OF A FINAL CLARIFIER

5. Report Date

6.

8. Performing Organization Report No.

7. Author(s)
Robert W. Agnew

10. Project No.
17090 FJW

9. Organization
Rex Chainbelt Inc.

11. Contract/Grant No.
14-12-194

13. Type of Report and Period Covered

12. Sponsoring Organization

15. Supplementary Notes

16. Abstract

An experimental testing program was carried out on final clarifiers at three treatment plants in order to provide a set of data for formulation and testing of a mathematical model for sludge compaction and solids separation performance. Multiple regression was used to evaluate the empirical coefficients in power functions for expressing the compaction and solids separation performance. The concentration of solids in the final effluent was related to overflow rate, BOD loading, mixed liquor suspended solids, and detention time. Maximum return sludge concentration was related to volatile fraction of mixed liquor suspended solids and BOD loading.

17a. Descriptors

*Settling Basins, *Mathematical Models, *Activated Sludge, Mathematics, Regression Analysis, Waste Water Treatment.

17b. Identifiers

17c. COWRR Field & Group

18. Availability

19. Security Class. (Report)

20. Security Class. (Page)

21. No. of Pages

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